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THE POSSIBLE IMPACT OF VESSEL WAKES ON BANK EROSION.(U)

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LEVEL II

THE POSSIBLE IMPACT OF VESSEL WAKES ON BANK EROSION

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FINAL REPORT

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16. Abstract

The purpose of this report is to provide a summary of the knowledge available on vessel-generated wake, and the possible impact of this vessel wake on bank erosion. A literature survey was conducted to identify the various causes of bank erosion along waterways. A summary of the various natural effects and possible vessel effects is provided.

Recession of waterway banks involves a large number of effects. The physical and chemical nature of the channel's water, the materials forming the bank, and the groundwater may increase the soil's erodibility by formerly noneroding water currents, wind waves, or vessel wakes.

No computational methods exist for linking a vessel with a chosen hull shape, traveling at a chosen speed in a channel of chosen depth and chosen cross-sectional area and shape with banks of chosen height and materials, to a predicted occurrence of erosion.

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METRIC CONVERSION FACTORS

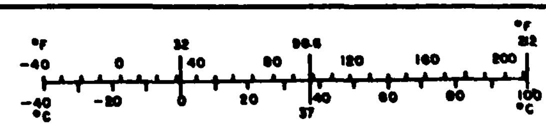
Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in ≡ 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.

Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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CONVERSION FACTORS, U.S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U.S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	by	To obtain
inches	25.4	millimeters
	2.54	centimeters
square inches	6.452	square centimeters
cubic inches	16.39	cubic centimeters
feet	30.48	centimeters
	0.3048	meters
square feet	0.0929	square meters
cubic feet	0.0283	cubic meters
yards	0.9144	meters
square yards	0.836	square meters
cubic yards	0.7646	cubic meters
miles	1.6093	kilometers
square miles	259.0	hectares
knots	1.852	kilometers per hour
acres	0.4047	hectares
foot-pounds	1.3558	newton meters
millibars	1.0197×10^{-3}	kilograms per square centimeter
ounces	28.35	grams
pounds	453.6	grams
	0.4536	kilograms
ton, long	1.0160	metric tons
ton, short	0.9072	metric tons
degrees (angle)	0.01745	radians
Fahrenheit degrees	5/9	Celsius degrees or Kelvins[1]

[1]To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use formula: $C = (5/9)(F - 32)$.

To obtain Kelvin (K) readings, use formula: $K = (5/9)(F - 32) + 273.15$.

SYMBOLS AND DEFINITIONS

A	cross-sectional area of a channel
C	cohesion
D	hydraulic depth, A/T
D	propeller diameter
d	water depth
d_b	depth of water over channel berm
d_c	minimum depth at the critical condition
F	fetch length
$\mathbf{F}$	Froude number
F_s	factor of safety for slope stability
g	gravitational acceleration
H	wave height
H	bank height above channel bottom
H_t	depth of tension cracks
H_w	height of water before a slope
H_w'	height of water table within a slope
h_p	shaft horsepower
I_w	Plasticity Index, defined in Figure 7
K	ration of critical shear force for a bank to critical shear force for a horizontal surface
K	a coefficient
LL	Liquid Limit, defined in Table 1
L_w	Liquid Limit, defined in Figure 7

SYMBOLS AND DEFINITIONS--Continued

N_{cf}	effect coefficient, Janbu slope stability, Figures 11 thru 15
PL	Plastic Limit, defined in Table 1
P_d	effect coefficient, Janbu slope stability, Figures 11 thru 15
P_e	effect coefficient, Janbu slope stability, Figures 11 thru 15
p	cotangent of channel sideslope
q	unit surcharge on top of slope
r_A	blockage ratio
r_d	depth ratio = $(d-d_c)/d$
S	salinity, defined in Figure 10
SL	Shrinkage Limit, defined in Table 1
s	shear strength of soil
T	wave period
T	width of water surface
U	wind speed
V	velocity of propeller jet
V_c	critical velocity, defined in Figure 10
V_c	ambient current velocity
V_{max}	maximum velocity from propeller jet on the channel bottom near the bank
V_o	initial velocity of propeller jet immediately behind the propeller
V_r	return current velocity
V_s	vessel speed

SYMBOLS AND DEFINITIONS--Continued

w	nominal water content, defined in Figure 10
X_O	effect coefficient, Janbu slope stability, Figures 11 thru 15
Y_O	effect coefficinet, Janbu slope stability, Figures 11 thru 15
α	slope angle of the bank
Δh	water level drawdown
θ	angle of repose of bank material
$\lambda_{c\phi}$	dimensionless slope parameter
μ_e	effect coefficient, Janbu slope stability, Figures 11 thru 15
μ_q	effect coefficient, Janbu slope stability, Figures 11 thru 15
μ_w'	effect coefficient, Janbu slope stability, Figures 11 thru 15
σ	stress normal to sliding surface
ϕ	angle of internal friction, $\tan\phi$ = friction coefficient

I. INTRODUCTION

The purpose of this report is to provide a summary of the knowledge available on vessel generated wake, and the possible impact of this vessel wake on bank erosion. A literature survey was conducted to identify the various causes of bank erosion along waterways. A summary of the various natural effects and possible vessel effects is provided.

Erosion of banks along navigable waterways is a continuing problem, which can require substantial maintenance of the waterway banks, and may result in damage to adjoining property. The causes of bank erosion are many, yet very site specific, and each must be considered in order to develop means of mitigating the problem.

A variety of natural bank materials will be found along waterways in the United States. Waterway banks may vary from loose sand or silt to rock, and may or may not be vegetated. Where vegetation exists, it may vary from grass to trees. Many of these banks have a high natural resistance to erosion, while others are highly vulnerable to natural and man-induced erosion. Dredged cuts for canals (e.g., the Intercoastal Waterway) may be especially vulnerable, particularly where the banks have a steep slope.

Where steep banks exist, instability of the bank material may be aggravated by natural ground water and seams of varying material within the strata of the gound. Ground water may saturate seams of material to cause sliding, and may also cause piping from the exposed face of the bank.

Other natural causes contributing to bank erosion include currents, waves, and ice. Currents may be particularly strong along river banks

and tidal inlets, and may cause some scouring of the banks. Waves may include both wind waves and swell. Swell would be limited to channel connected to the open sea, while wind waves may occur along any reach of channel. It is expected that the wind waves would be more damaging to channel banks than swell, and the height and period of these waves will depend upon the length of channel reach and the magnitude and direction of the prevailing winds. Wind blowing across a narrow channel will have little effect, but wind blowing along the length of a long reach may produce relatively large waves traveling nearly parallel to the shore and, consequently, waves which would be expected to contribute significantly to bank failure. Floating ice is not normally damaging to waterway banks because ice usually occurs when the banks are frozen; but in some instances ice may damage banks by scour if currents are moving ice parallel to the banks.

Effects from vessels on bank erosion include ship waves (generally propagating from the bow and stern of the vessel), the stern transverse wave, the return current, the slope-supply flow, and the propeller jet. These effects are influenced by the vessel's hull design, displacement compared to the channel's cross-section, speed in relation to tidal or river currents, and distance from banks and other vessels. The stern transverse wave results from the drawdown of water alongside the vessel as displaced water in front of the vessel flows around the vessel to the stern. In a narrow channel, where the blockage ratio is high, the stern transverse wave will have the appearance of a moving hydraulic jump, propagating along the channel at the speed of the vessel. Depending on the distance between the vessel and the channel bank, waves propagating from the bow of the vessel may coincide with the stern transverse wave and amplify the wave height at the bank.

The motion of a vessel moving along a channel will cause a return current in the channel, moving in the direction opposite to vessel motion, because of the water displacement in front of the vessel. In addition, currents are created along the channel bank, flowing from the area behind the vessel into the drawdown area alongside the vessel. These latter currents are known as slope-supply flow, and may cause erosion of bank material with the material moving in the direction of vessel motion.

Additional problems related to vessel movement may be caused by the propeller jet, particularly when vessels are navigating close to a channel bank. Wash from a propeller jet may occur well below the water surface, and may contribute to undermining bank protection such as riprap. Effects of the propeller jet may be more pronounced in channel bends, and near lock entrances or other locations where engines run for relatively long periods near the same point in the channel.

Other effects of vessels may include vessel impact on the channel bank, with resulting damage to the bank and bank protection, and damage resulting from vessels tied up to trees along the channel bank. In the case where trees are used for mooring vessels, the trees may be killed and eventually pulled into the channel, resulting both in bank failure and the creation of snags in the channel.

The relative effect of any erosion cause is highly site specific, and can vary significantly depending on such factors as channel width, blockage ratio, importance of normal wind generated waves, bank materials, ground water inflow, flood currents in rivers, etc. For example, wake-caused erosion has been of relatively high importance in the Suez Canal where banks are low and of sandy material, and relatively narrow; on the other hand, extensive studies on the Ohio River (U.S. Army Engineer

Division, Ohio River, 1977) have shown wake-caused erosion to be insignificant compared to bank slumping resulting from rapid drawdown of the river adjacent to saturated banks after a flood event. Thus consideration of the relative importance of vessel wake or any other factor that may cause erosion must be looked at on a case-by-case basis.

In attempting to combat these erosive forces, various materials are used for bank protection including sod, riprap, grout-filled fabric mattresses, gabions, concrete mattresses, bulkheads and concrete soilcement, or asphalt paving. These different types of bank protection have varying degrees of resistance to natural processes and to vessel effects. Failure of bank material may result from slope failure of the bank, undermining of the bank protection, or actual movement of the bank protection (e.g., riprap stone) by wave and current action.

At sites where vessel wake appears to be a predominant cause of bank erosion, protection of channel banks by regulating vessel traffic may be difficult due to the variation of wake conditions generated by different types of vessels under different flow conditions, and the seasonal variation in conditions along the channels. Higher wake effects may result from high-speed small craft with cruiser shaped hulls than from slow barge tows, so that wake limits (i.e., no wake zones) may be easier to specify than speed limits, but these limits may be difficult to enforce. Seasonal variations may cause high water conditions at particular times during the year. These high water conditions may allow vessels to approach closer to channel banks, and may expose upper portions of channel banks to wave and current action. Some restrictions could be placed on navigation close to the channel banks.

II. CHANNEL BANK MATERIAL

Erosion of channel banks due to waves is generally observed to be an ongoing cyclical process as shown in Figure 1. The process most logically begins with erosion of the toe of the bank which may be somewhat above, at, or below the mean water level. While a quantitative prediction of this erosion is difficult to ascertain the effects of it are easily observable. Toe erosion causes a general slope steepening which eventually creates a slope instability and failure. The removal of the slope failure's talus completes the cycle and permits fresh erosion of the new toe of the bank. Erosive currents have the same effect.

A channel bank's stability against sliding and its erodibility by waves and currents are both major factors which may influence the rate of bank recession. Investigating the causes of a particular case of bank recession involves determining whether the bank instability is a result of changing forces within the bank, as discussed later in this chapter, or of erosion at the toe of the bank, as discussed below. If a slope stability analysis demonstrates that bank material and groundwater effects are not responsible for the recession, an investigation of erosion must be made, asking what wave or current conditions could initiate erosion, what rate of erosion could occur, what volume of erosion would be required to initiate sliding, and what volume of sliding could occur. Each step in the analysis considers various factors which depend on the properties of the bank material, which may be rock or soil.

While some types of rock are susceptible to erosion, most problems occur with banks of clayey, silty, or sandy soils. In general, bank soils fall into two classifications: cohesive and cohesionless. Cohesive soils are clays, which consist of fine particles of chemically

active minerals that create strong bonds between particles. The chemical activity of the clays makes the analysis of the behavior of cohesive soils complex. Cohesionless soils are coarser material, i.e., silts, sands, and gravels, which have no chemical or electrochemical bonding between particles themselves.

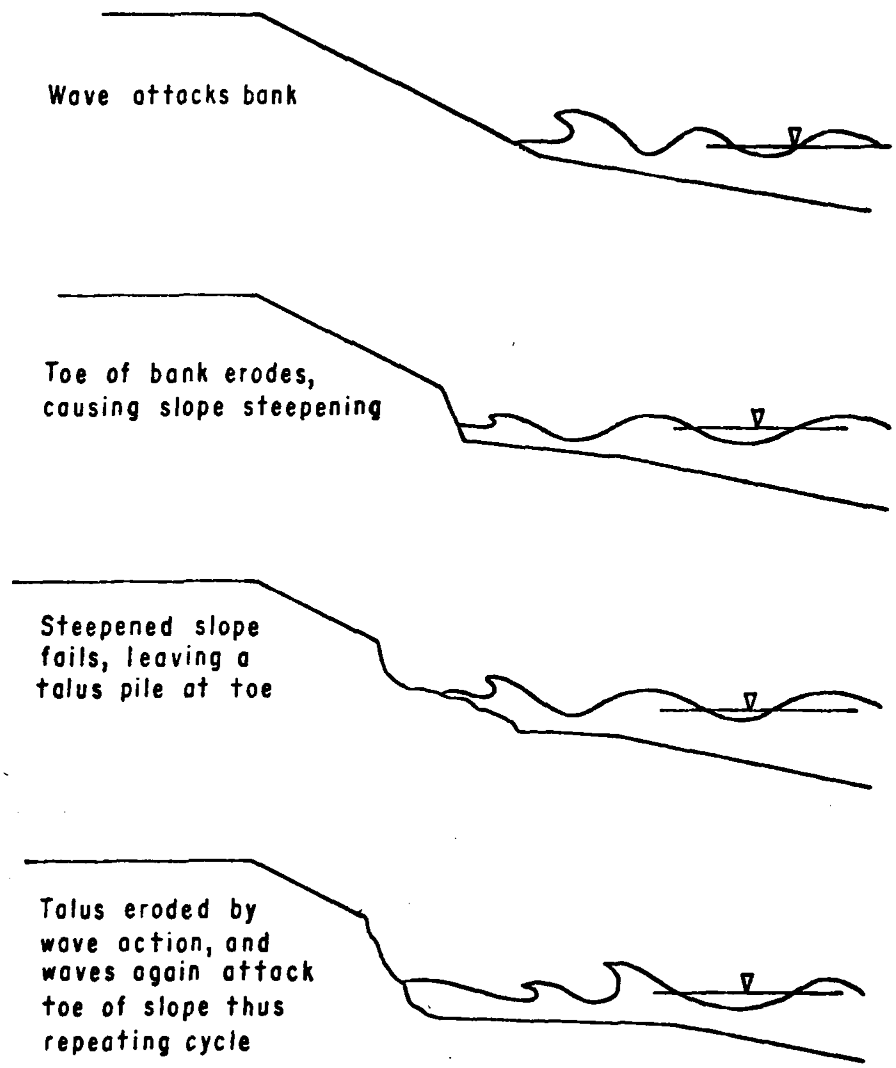


Figure 1. Wave-caused bank erosion cycle

Erodibility and slope stability depend on the different properties for cohesive and cohesionless soils, and on a combination of the properties for soils that are a mixture of coarser and finer types. The variety of soils makes the proposal of general rules impractical. The following sections on bank erodibility and slope stability will discuss the many factors involved and, where possible, will describe simple methods of predicting the behavior of a bank.

1. Bank Erodibility.

A bank will not erode if forces acting on the soil particles are in balance. These forces are gravity, buoyancy, lift, drag, and cohesion or interlocking. Gravity draws the particle downward, into the bank or along the face of the bank, depending on the slope, while buoyancy pulls the particle upward. Lift, generated by water flowing over the surface of the particle, tends to pull the particle out of the bank, while drag forces push it along the face of the bank or, in the case of a porous bank with waves breaking onto it and backrush flowing out of it, push the particle into or out of the bank. Even when waves are not a factor groundwater seeping out of the bank can pipe particles from the exposed surface. The surface tension of water droplets trapped between particles above the water line, cohesive forces caused by electrochemical attraction between clay minerals or coatings of organic material, and interlocking between angular particles all tend to hold the particle in place in the bank.

The magnitude of the forces is related to the geometry of the bank, the flow of the eroding fluid, and the engineering properties of the

soil. The geometry of the bank, especially the slope near the water line, determines the extent to which gravity tends to pull the soil particle down the bank, the force increasing with steepness. The height of the water level in the bank determines the influence of groundwater. Changes in the channel's water level, especially rapid cyclical changes, such as floods and tides, can effect bank stability and erodibility. The chemical properties of the eroding fluid interact with those of clays to affect the strength of cohesive bonds. The magnitude of other forces varies with the characteristics of the soil itself, for reasons explained in the following sections.

a. Cohesionless Soils. For cohesionless soils, soil properties such as particle size range, gradation, i.e., the distribution of sizes within the range, and the degree of burial of the particles in the bed determine the magnitude of the lift and drag forces for a given velocity. The mineral constituents of the particles determine their density and weight. The relative density, a measure of the closeness of the packing of cohesionless particles, along with the shape and gradation of the grains, determines the amount of interlocking. The water content, defined as the ratio of the weight of water in a soil sample to the weight of the oven-dried soil contained in the sample, determines the weight of the soil and the bonding due to the surface tension of interparticle water, especially in silts and sands.

(1) Effect of Grain Size and Flow Velocity. The primary factors determining initiation of scour for a cohesionless soil are grain weight and surface area, both of which are characterized in part by grain

size. Figure 2, from Keown, et al. (1977) and Kolb (1956) shows the relationship between the grain size and the critical velocity at the bottom required to begin erosion of a horizontal bed composed of fine sand to large gravel. Notice that the titles for different size ranges, according to two of several classification systems in use, are given on the horizontal axis with the grain size increasing from right to left. The velocity at a point on the bottom or bank is difficult to determine. When data exists, the relationship between the bottom or bank velocity and the more easily measured mean flow velocity can be derived, as shown in the inset in Figure 2, and can be used to determine the possibility of erosion from flow or tidal currents. Velocities produced by vessel effects are discussed in later sections of the report. Notice that, in a mixture of grain sizes, the smaller particles may be removed by a flow that leaves larger particles, such that if sufficient large particles exist to form a stable lattice, they may form an armor layer protecting the finer material below.

(2) Critical Shear Force. The best-established relationship for the initiation of motion is between grain size and the critical shear force, the force required to initiate particle movement, generated by water movement along the surface of the bottom or banks and acting in the direction of flow. The shear force at a point on a bank or bottom is a function of the square of the velocity at the point, and, due to the difficulty of measuring a typical velocity in the field, is difficult to evaluate. The critical shear force, and corresponding

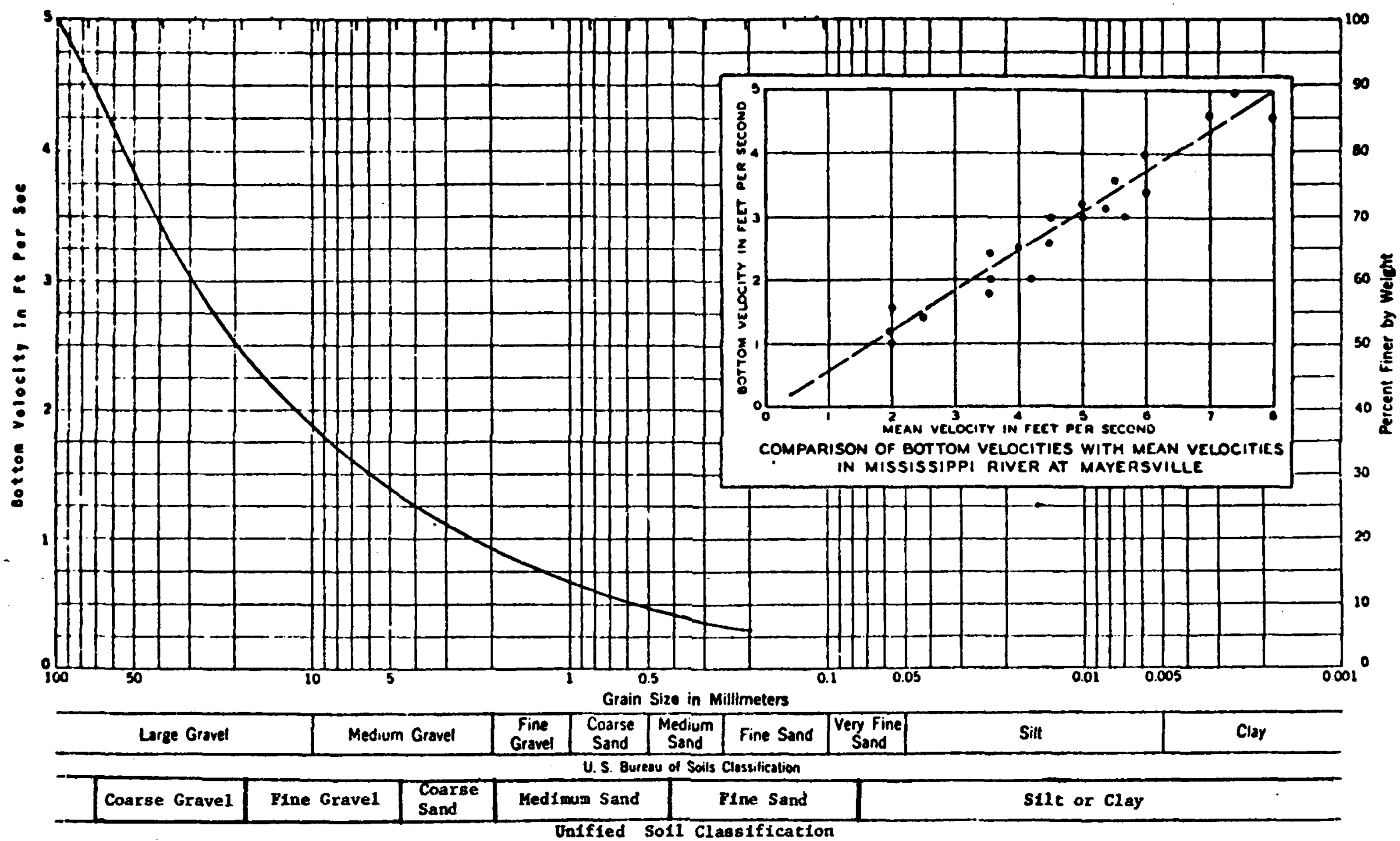


Figure 2. Bottom velocities for initiation of movement versus grain size of fine sand to large gravel. (from Keown, et al. 1977, and Kolb, 1956)

critical velocity initiating erosion, is less for banks than for the bottom as a result of the contribution of gravity to the cohesionless particles' instability. The amount of reduction in the critical shear force is a function of the bank slope and cohesionless soil's angle of repose.

(3) Angle of Repose. The angle of repose of a cohesionless sand or gravel, as defined by Lane (1955) and as used in Figures 3 and 4, is the angle between a horizontal plane and the face of a freestanding, loosely-poured pile of the soil. By Lane's definition, angle of repose corresponds to the more generally used term "angle of internal friction" for a cohesionless soil in its least dense state. The angle of internal friction is discussed further in the section on slope stability.

When a bank is at the angle of repose, the forces maintaining cohesionless particles in the bank have balanced the gravity force pulling particles down the bank, and the imposition of erosive forces is likely to start erosion. The angle of repose increases with increasing particle angularity and corresponding interlocking. The angle varies as the water content changes, first increasing with increasing water content up to about 10 to 15 percent, then decreasing as additional water content reduces, then eliminates, the surface tension effect of pore water. When, with further increases in water content, downslope seepage occurs, it is an added force pushing the particles outward. As an illustration, Figure 3, from Lane (1955), presents the approximate angles of repose of medium to coarse gravel with varying degrees

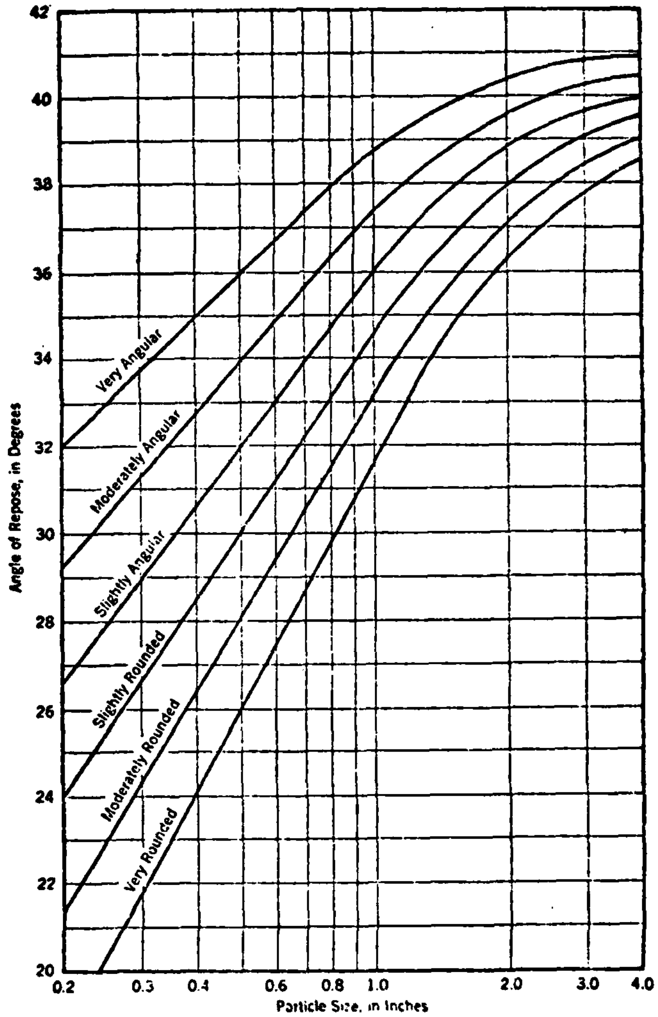


Figure 3. Angle of repose of noncohesive gravel-size material. (from Lane, 1955)

of particle angularity. The angles of repose are rough averages from wet and dry gravel piles in air and from submerged gravel piles. Notice the significant effect of grain shape on the stability of a bank.

(4) Effect of Bank Slope. The relationship between the critical shear forces for channel banks and channel beds, where the erosive force is acting along the length of the channel, is illustrated in Figure 4 (Lane, 1955) for the gravels described in Figure 3. K is the ratio of the critical shear force for the bank to the critical shear force for a horizontal surface, and is calculated from

$$K = \cos \alpha \sqrt{1 - \frac{\tan^2 \alpha}{\tan^2 \theta}} \qquad (1)$$

where θ is the angle of repose of the bank material and α is the slope angle of the bank. Notice that Figure 4 applies to forces acting along the bank, such as tidal or flow currents, and not to waves or currents acting directly up or down the bank. The Figure also applies only to loosely compacted gravel, for a tightly compacted gravel of a given size can form stable banks with slope angles steeper than the angle of repose given for the size in Figure 3. Lane notes that the data used for Figure 3 was widely scattered. If the angle of repose from Figure 3 is in error by a few degrees the effect on the value of K from Figure 4 is significant. This must be taken into account when using Figure 4.

(5) Evaluating a Cohesionless Soil. The grain size analysis of a sample of a cohesionless soil, performed using techniques described in any soil mechanics textbook, can be used to determine the mean

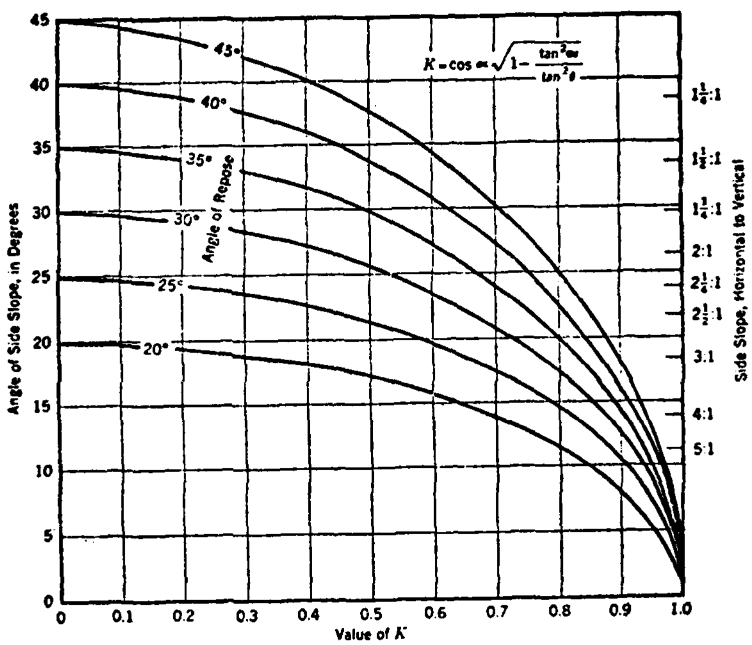


Figure 4. Relationship between side slope and K for gravel-size material. (from Lane, 1955)

grain size. With the mean grain size, Figure 2 may be used for sand and gravel and Figure 5 for finer material to estimate the critical velocity for a horizontal bed, a rough approximation of the maximum critical velocity for a bank. If the critical shear force for the mean grain size can be found in the literature, the angle of repose of several samples of the bank material may be measured using techniques discussed in soil mechanics textbooks, then used in Equation 1 with the natural bank slope to estimate the reduced critical shear force for the material in the bank. The actual critical velocity for the bank material may be lower than the estimates. Laboratory testing of an undisturbed sample of the soil would be necessary to determine an accurate critical velocity.

The erosion rate of a cohesionless soil must be estimated from field measurements of the volume of erosion over time, or from laboratory tests. This usually must be done by a hydraulic engineer familiar with the behavior of local sediments. Unless the natural conditions are much less severe than those induced by ships, or erosion can be traced to isolated extreme events, such as the passage of a single ship removing enough material to start a landslide, separating the erosion rate due to ship effects from the rate due to currents or wind waves may be very difficult. Compared to other materials, the erosion of fine sands and silts, the most troublesome materials, occurs rapidly once movement is initiated.

b. <u>Cohesive Soils</u>. A soil exhibiting cohesion may be almost entirely clay or, more typically, a mixture of clay with silts, sands, or gravels. For such a soil, a decrease in mean particle size, signifying an

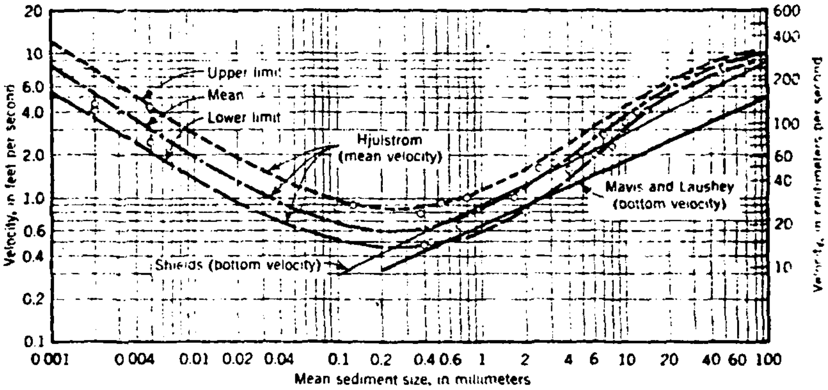


Figure 5. Critical water velocities for sediment versus mean grain size. (from Vanoni, 1975)

increase in clay content, is accompanied by a decrease in the lift and drag tending to displace the particles and in the weight maintaining the particle in position, but by an increase in the strength of cohesive forces. In a given mixture of clay and other soils, the cohesion varies with the type of clay, the structure and content of the soil, and the chemistry of the ground water and eroding fluid. As the soil becomes predominately clay, the lift and drag depend more on the structure than the size distribution of the clay, and the effect of the decrease in weight is overcome by the increase in cohesion.

(1) Effect of Grain Size and Flow Velocity. The water velocities required to cause movement of mixtures of silt and clay in natural stream beds are compared to those for sand and gravel in Figure 5 (from Vanoni, 1975). Notice that the grain size increases to the right. On the left half of the curve, for fine sand to clays, the maximum critical velocity for a given mean particle size is twice the minimum critical velocity. This reflects the complexity of the factors involved, including differences in the way researchers define initiation of erosion, and variations in particle size distribution for a given mean particle size, as well as the variables characteristic of the different clays. At the critical velocity starting movement, the erosion rate for a clay soil may be very low. The erosion rate must be determined separately for each different type of clay and mix of clay with other particles sizes. For some clay soils the relationship between velocity and the erosion rate may be of more importance than the critical velocity.

(2) Bonding of Clay. Clay is composed of compound minerals produced by the chemical weathering of the minerals in silt and sand. There are several categories of clay minerals, each with different properties. Most clay minerals form plate-like particles which exhibit strong electrochemical forces, negatively charged on the face and positively charged on the edges. Water molecules and cations are attracted to the face and become bound in a structure resembling ice. Farther from the face, water and cations are bound in a more random order. The bound water and cations form the diffuse double layer shown in Figure 6, taken from Sowers and Sowers (1970).

Bonding between clay particles occurs when the double layers between the particles join or when a plate's edge meets another plate's face. The edge-to-face bonds are the strongest, forming a rigid soil structure. The double layers act like a viscous "glue", allowing movement between the plates without loss of strength. This ability to deform without cracking is called plasticity, and this property is one of the primary differences between the behavior of clays and silts. For some clay minerals, as the soil's water content increases over a wide range, the thickness of the double layer increases without loss of bonding and the soil expands as its water content increases and shrinks as it decreases. These clays are called highly plastic, and, if the double layer's thickness changes by a large proportion of the original thickness, they are also called highly expansive. As the distance between particles increases, the bonds between particles weaken. Beyond a certain point the clay loses its strength and becomes liquid.

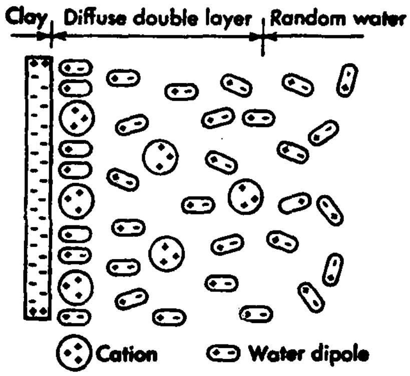


Figure 6. Absorbed water and cations in diffuse double layer at the surface of a clay particle. (from Sowers and Sowers, 1970)

As the water content of a clay decreases, or as it is compacted by outside forces, particles move closer together and the edge-to-face bonds are more likely to occur.

(3) Atterberg Limits. The relationship between a cohesive soil's water content and the soil's engineering properties is described in terms of the Atterberg Limits, as explained in Table 1 from Sowers and Sowers (1970). When the water content is at or above the Liquid Limit, the soil will flow under a very small stress. Between the Liquid Limit and the Plastic Limit the soil will deform without cracking, but for a water content below the Plastic Limit cracks will form under stress. As the water content is reduced below the Shrinkage Limit, decreases in volume cease. The ability of the soil to absorb water into its structure is characterized by the Plasticity Index, the difference between the Liquid and Plastic Limits. The combination of Liquid Limit and Plasticity Index is used to determine whether a fine-grained soil, typically a mixture of silt and clay, behaves primarily as a silt or as a clay. Figure 7, from Graf (1971) and Terzaghi and Peck (1968), presents the classification system. For a soil that is classified as a clay, the combination of properties plotted on the figure is an indication of the type of clay mineral comprising the soil. The Atterberg Limits are measured in a soils laboratory using methods described in any elementary soil mechanics textbook, such as Sowers and Sowers (1970) or Krebs and Walker (1971).

Figure 8, from Gibbs (1962), presents the relationship between Atterberg Limits and erosion characteristics determined in laboratory erodibility tests of various types of clay. In the figure, "resistance

Stage	Description	Boundary or Limit
Liquid	A slurry; pea soup to soft butter; a viscous liquid	
		Liquid limit (LL)
Plastic	Soft butter to stiff putty; deforms but will not crack	
		Plastic limit (PL)
Semisolid	Cheese; deforms permanently but cracks	
		Shrinkage limit (SL)
Solid	Hard candy; fails completely upon deformation	

Table 1. Atterberg Limits. (from Sowers and Sowers, 1970)

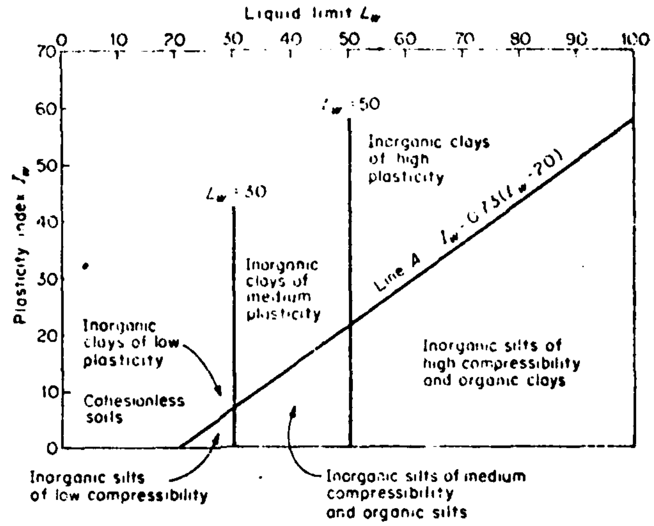


Figure 7. Plasticity chart. (from Graf, 1971, and Terzaghi and Peck, 1968)

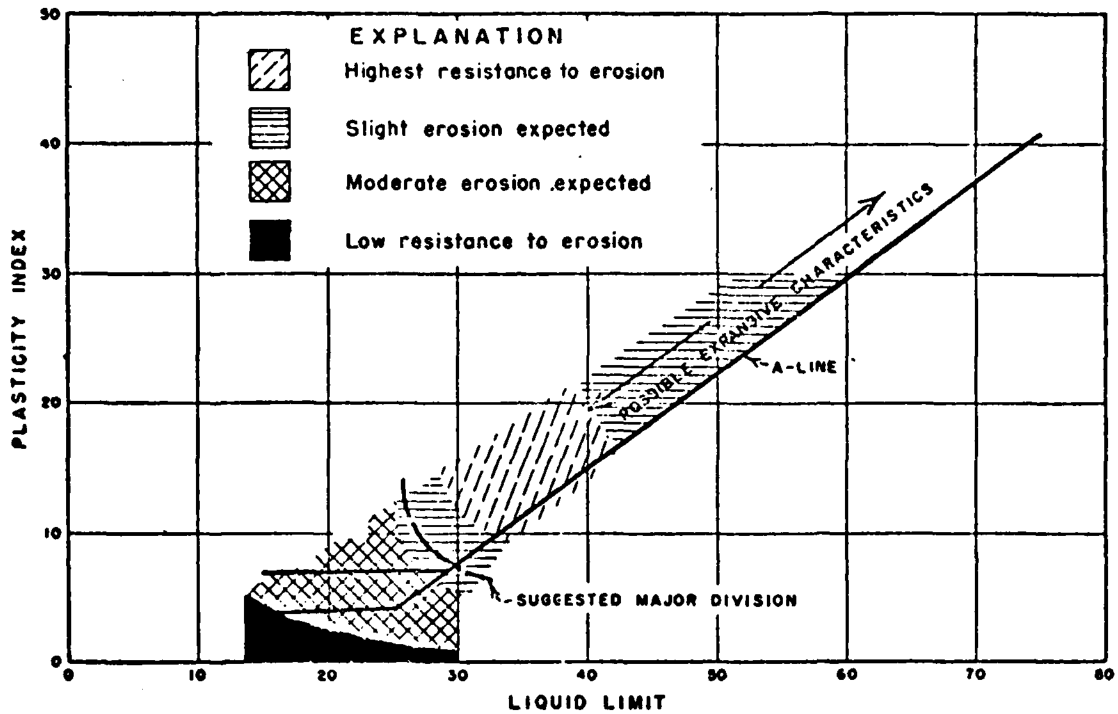
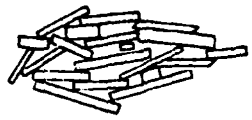


Figure 8. Relationship between Atterberg Limits and erosion characteristics. (from Gibbs, 1962)

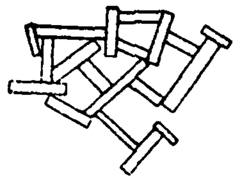
to erosion" refers to the critical shear force required to initiate movement, high resistance corresponding to a high critical shear force. From a comparison of Figures 7 and 8, notice that medium plasticity clays are the most resistant.

(4) Types of Clay Soil Structures. The structure of a clay soil lies between the extremes of completely random and completely oriented, called flocculant and dispersed, respectively, from the way the structure was formed. In the oriented structure, the plate-like clay particles are stacked face to face, as shown in Figure 9a from Krebs and Walker (1971). In the flocculant structure, the arrangement of the particles is more random, with larger pore spaces between particles but more face-to-edge bonds, as shown in Figure 9b, also from Krebs and Walker (1971). Figure 9c, from Partheniades and Paaswell (1970), illustrates an "aggregate" or "packet" structure, where clay particles form a tightly-bonded oriented or flocculant structure within flocs, surrounded in the figure by solid lines, and the flocs become loosely bonded to form packets or aggregates, surrounded in the figure by dashed lines, which loosely bond to one another to form the soil structure. The void ratio of a cohesive soil, defined as the ratio of the volume of voids to the volume of solids in a sample of a soil, is determined by the type of structure.

(5) Effect of Clay Soil's Structure. The erosion rate and, in some cases, the critical velocity for a clay soil both decrease with increasing orientation, making the erodibility dependent on whichever effect predominates. As explained by Paaswell (1974), the oriented structure presents less resistance to flow, decreasing the shear force



(a) Idealized dispersed or oriented structure.
(face-to-face contact)
(from Krebs and Walker, 1971)



(b) Idealized flocculated structure
(edge-to-edge contact)
(from Krebs and Walker, 1971)

(c) Aggregate structure.
(from Partheniades and Paaswell, 1970)

Figure 9. Schematic diagrams of particle orientation in types of clay soil structures.

on the soil. When the soil erodes, only individual particles are removed due to the weak bonds or repulsion between particle faces, but this weak bonding also decreases the clay's critical velocity. The surface of the flocculant structure is rougher, producing more shear force. Unless a flocculant soil, like that in Figure 9b, has been compressed after placement, under erosive forces the particles tend to act in groups, each group behaving like a large grain of soil with weak bonds or only a few strong edge-to-face bonds to the groups around it. The strength of the few strong bonds and the weight of the group of particles may require that the velocity to begin erosion of the flocculant structure be higher than for the dispersed structure, but, once erosion begins, the poor bonding between groups of flocculant particles allows the groups to be rotated and removed from the soil's surface. This removal in groups makes the erosion rate for flocculant clays higher than that for dispersed clays.

(6) Formation of Structure. The soil structure is a product of the type of clay mineral in the soil, the percentage of clay in the soil, and the history of the soil. The type of clay mineral determines the strength of the charge on the plate-like grains, and that is the primary determinant of the properties of the bonding between particles. The bonding properties determine the plasticity, expansive characteristics, and floccing characteristics, and also govern the intensity of the effects a particular clay has on a soil of mixed particle sizes. Except for highly expansive clays, as the percentage of clay mixed with silt and sand increases, the cohesiveness of the soil increases without a disproportionate decrease in density, and the erodibility decreases.

The history of the soil includes the deposition of the soil, the history of disturbances weakening the soil structure, and the stress history of the soil.

In areas where ship and barge channels occur, most soils have been deposited in their present location by settling out of water. For some types of clay minerals, the types of bonds formed between particles at the time of their deposition depend on the types and concentrations of cations in the water carrying the particles, generally measured as the salinity of the water. For these clays, as salinity increases, the attraction between particles suspended in the water increases until the particles form groups, or flocs, with the particles tending to be joined edge-to-face. When the flocs settle out, the soil has a flocculant or an aggregate structure, as in Figures 9b and 9c. If the salinity of the water is low, or if the clay is insensitive to the salinity, the particles stay apart, or disperse, and settle out individually, as in Figure 9a.

Soils that have been recently disturbed or that are newly deposited are weak and most erodible. Since the formation of the double layer is a chemical process that takes time, clays may increase in strength if not disturbed further, a property called thixotropy. Highly plastic clays with a flocculant structure, if not remolded to the point that the flocculant structure is destroyed, will, in an environment promoting a flocculant structure, gain a significant amount of erosion resistance as the formation and strengthening of bonds reorients particles and increases the close edge-to-face bonding. Any stress that tends to compress the soil will consolidate it, decreasing the distance

between grains and increasing the strength of bonds, unless the soil is very porous and contains large amounts of pore water that cannot drain out if the compressive force is applied too rapidly. The most common cause of compression is the deposition of overburden with a corresponding increase in weight on the soil. Compaction of a clay soil with an aggregate structure forces the aggregates and flocs into a denser arrangement, or, if the force is great enough, breaks up the flocs and aggregates to form a generally flocculant structure. According to Partheniades and Paaswell (1970), neither of these effects has a significant influence on erodibility. Compaction of a soil that already has a flocculant structure increases the orientation of the soil particles and, by forming a more dispersed structure and increasing the number of edge-to-face bonds simultaneously, increases the erosion resistance of the soil.

(7) Causes of Structural Changes. Except for compaction, the effect of mechanical disturbance on most clays is to loosen the structure and increase the erodibility. Soil in a bank that has been newly deposited out of water, altered by sliding, cut by dredging, or formed of uncompacted material at the edge of a landfill is likely to be less resistant to erosion than soil that has been in place in a stable bank for several years. Changes in pore water and eroding fluid properties may alter the structure also, increasing the erodibility of the clay soil in banks with histories of stability.

(8) Effect of Clay Soil's Water Content. The relative volume of pore water, measured as water content, determines the amount of water available to form a clay's double layer. Soil underwater and a few feet

above the water line usually is saturated and has a water content above the Liquid Limit. High waves or rises in the water level may increase the water content of soil ordinarily not saturated. For clay soils of low plasticity the water content has little effect on the structure or erodibility of the soil, but for medium to high plasticity clays the water content of the surface layer of soil is a primary determinant of erodibility, the effect depending, to some extent, on the state of compaction of the soil and other factors. For water contents in the range of the soil's Liquid Limit and above, the increase in inter-particle spacing and decrease in bonding for medium to high plasticity clays, and especially for highly expansive clays, increase their erodibility. Figure 10 from Gularte (1978), shows the effect of moisture content on the velocity causing erosion of a horizontal bed composed of a thoroughly remolded mix of silt and medium plasticity clay, with the clay assumed to be forming a dispersed structure. Although Figure 10 shows the velocity increasing before it decreases, according to a summary by the Task Committee on Erosion of Cohesive Material (1968) (part of the Committee on Sedimentation, Hydraulic Division, ASCE), most researchers have reported a steady decrease with increasing water content for other clays. The erosion rate, most researchers report, decreases to a point, probably around the clay's Liquid Limit, then increases with increasing water content.

Extreme changes in the history of a clay's water content are also important. Dessication, the severe drying out of a soil, decreases the water in the double layer, drawing the particles together. The erosion resistance of high plasticity clays and some medimum plasticity clays

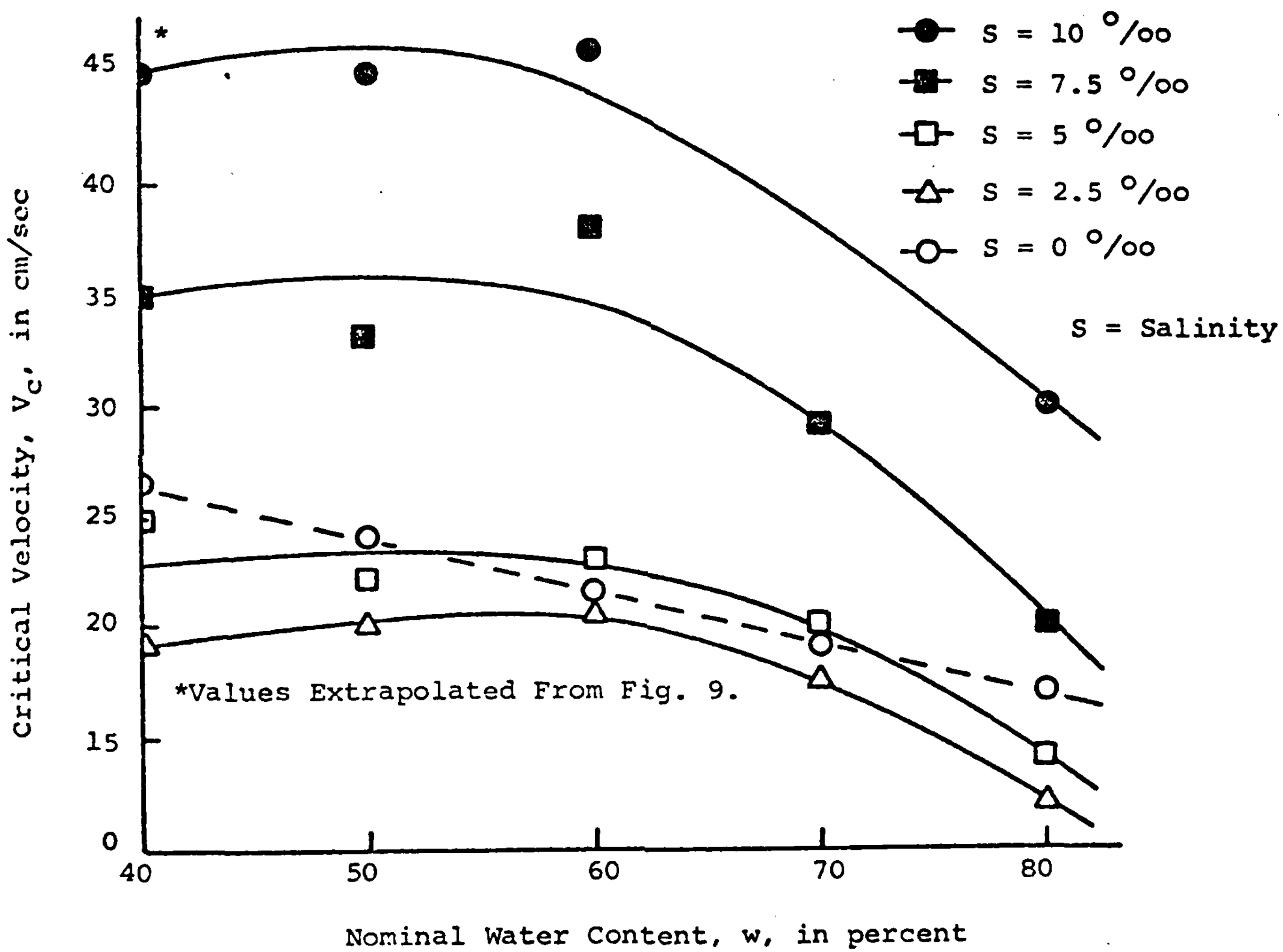


Figure 10. Critical water velocity as a function of the water content of a soil of medium plasticity as affected by the salinity of the pore and eroding fluids. (from Gularte, 1978)

is increased by any amount of drying, but the Bureau of Reclamation (1961) reports that shrinkage cracks form in silty low-plasticity clays, weakening them. When the surface of a dessicated fine-grained soil becomes saturated, bubbles of air trapped in the pores may become compressed and counteract the cohesive forces. If this weakens the surface layer of soil sufficiently, the surface may disintegrate and erosion at a high rate may occur at an unusually low velocity. This phenomena is called slaking. Dessication usually occurs only in the arid to semi-arid climate of the western U.S. and is unlikely to occur in navigation channels constantly filled with water unless the channel has been newly cut from dry ground.

(9) Effect of Clay Soil's Pore Water Chemistry. Just as the structure of a clay is affected by the chemical properties of the water in which the clay is deposited, changes in the chemistry of the pore fluid can alter the structure. As the salinity increases, and with it the availability of cations in the pore water, the strength of bonds increases and the erodibility of the soil decreases. This is illustrated by the increase in critical velocity shown in Figure 10. The acidity of the pore water, measured as pH, may have an effect on the erodibility of some clays, but the relationship is not well established and may be masked by the salinity effects.

(10) Effect of Eroding Fluid on Clay Soil. The salinity and pH of the pore fluid near the submerged surface of a soil is influenced by the properties of the eroding fluid. If the eroding fluid is more saline than the pore fluid, and especially if the cations in the eroding fluid are types that strengthen bonds, the erosion resistance of the surface layer of soil can improve. If the eroding fluid is less saline than the pore water, or the cations present are a type that

weakens bonds, the surface layer of soil may be weakened by the leaching or replacement of pore fluid ions, and the erodibility increased. When the salinity is less in the eroding fluid than in the pore fluid, the difference in salinity produces an osmotic pressure that draws groundwater out of the bank, creating an outward pressure on the surface layer that adds to the erosive forces. In some experiments, described by Arulanandan, et al., (1975) and discussed by Paaswell (1974), high plasticity clays eroded only if the eroding fluid was less saline than the pore fluid. For some soils, if the pore fluid is more acidic than the eroding fluid, the effect of salinity on erodibility may be affected, but the nature of the change, like the influence of pH in general, is not well defined. In Figure 10 the salinity and pH of the pore fluid and eroding fluid are identical.

As the temperature of the eroding fluid increases, the erosion rate of fine-grain soils increases, as reported by Partheniades and Paaswell (1970), Gularte (1976), and Kelly et al. (1979). With the increase in temperature the viscosity of the fluid decreases, causing an increase in turbulence and suspension of particles. The fluid temperature does not affect the critical velocity, but after the critical velocity has been attained or exceeded, the temperature affects the rate of erosion at a given velocity. According to Grissinger (1966), the erosion rate at 35°C (95°F) for some clays is twice the rate at 20°C (68°F). Other authors report similar effects at the lower temperature ranges found in nature. The temperature effect decreases in influence as the salinity of the eroding fluid increases.

(11) Causes of Fluid Changes. The practical effects of the relationship between a cohesive soil's erodibility and the properties of the pore and eroding fluids are complex and difficult to define for a particular soil. In the area of an industrial outfall the temperature, pH, and cation types and concentration of the channel water may be altered and may vary with time. Leaking sewage pipes and septic fields may change the chemistry of the pore fluids in a soil. The pump-down of wells in a fresh water aquifer near the coast may lower the level of the aquifer and allow the intrusion of salt water, or heavy rainfall may introduce large amounts of free water to the channel, upsetting the balance of salinity between the pore fluid and eroding fluid.

(12) Effect of Bank Slope and Groundwater Elevation. Cohesion can be strong enough to hold a vertical or undercut bank intact. Seepage flow from a high water table may wash soil out of a bank if the flow is strong enough to overcome cohesion. The bank slope and seepage flow have less effect on the erodibility of a cohesive soil than on the slope stability, as will be discussed in the sections on that subject.

(13) Evaluating a Cohesive Soil. The erosion of cohesive soils is, for the most part, still an unexplained phenomenon. The many factors discussed are known to influence erodibility, but the relative magnitude of the effects has not been determined. The data needed for evaluating the behavior of a cohesive soil are the grain size distribution, the Atterberg Limits, the void ratio, and the in-place water content of the soil, and the temperature, the pH, and the ion

concentration and content of the pore and eroding fluids. Numerous reports exist relating these factors to the critical velocity and erosion rates of specific cohesive soils. Application of the information in these reports to the situation at a given location will require the aid of a hydraulic engineer, preferably one familiar with the local soils.

c. Rock. Banks of hard rock, such as granite, do not generally present erosion problems, but weakly cemented conglomerates and rocks vulnerable to weathering may experience erosion. Porous rock, coquina limestone for example, and rock with significant jointing can undergo both chemical and mechanical weathering from exposure to organic acids in groundwater and to ice expansion. Chemical weathering is especially severe in tropical climates where biochemicals abound in the groundwater. Water waves and currents tend to erode the weathering products exposing fresh rock for more weathering. Clay shales, carbonaceous and lignitic shales, and agglomeratic tuff have been eroded by ship and wind waves in the Panama Canal according to the Canal Zone Department of Operations and Maintenance (1947). The material produced by the breakdown of rock will resemble one or more of the soil types, and its erodibility will depend on the factors affecting that type of soil.

2. Slope Stability of Channel Banks.

The evaluation of the existing slope stability and the estimation of what would be a stable slope profile for use in designing remedial measures are key steps in estimating the impact of allowing continuing wave erosion, and in determining the relative effect of erosion on past bank recession, in stabilizing the bank. To do this the engineer must analyze the balance between the actuating forces that tend to

cause failure, generally gravity or seepage related, and the residual strength of the soil to prevent failure. Generally, the actuating forces can be predicted quite accurately by existing techniques, however, the evaluation of soil strength is more difficult and requires very careful and extensive field and laboratory testing to achieve good results. Because of the cost of such testing is quite high, data collection along many channel banks will be inadequate to predict soil parameters very accurately. Instead properties are averaged between boring sites and rely on past experience to estimate reasonable strength values.

The analysis of slope stability can be accomplished by any of several methods, Wright (1969) reports on thirty-seven methods currently in use, however, several factors must be considered in selecting a method. First is the type of failure surface expected. The typical failure surface for banks which have been steepened by wave induced toe erosion will be a circular or compound surface passing through the toe of the bank. The occurence of silt strata within the soil will modify this but a circular surface may still be assumed for a preliminary analysis. Another consideration in selection is the occurence of special conditions, i.e,, submergence, surcharge, tension cracks, etc. The method used should provide for consideration of these. Finally the method should be consistent with the quality of data available. As mentioned above the soil strength data used in design will frequently not justify a detail analysis. When such detailed analyses are required, a geotechnical engineer should be tasked to undertake a plan of soil sampling and slope analysis adequate to the problem at hand. However, it is frequently desirable to obtain a preliminary estimate of bank stability for use in planning further actions and a simple method of analysis

which uses charts to assist the solution is presented below for this purpose.

The advantage of stability charts which assume simplified slope geometry and uniform soil conditions are their easy, rapid solvability. The Janbu stability chart method presented here is preferred because of its capacity to consider the many special situations typical of channel bank stability problems, i.e., tension cracks, partial submergence, variable water table location in slope, and surcharge loads. Other stability charts are presented in the literature, i.e., Duncan and Buchignani, 1975, Tenzaghi and Peck, 1967, and Winterkorn and Fong, 1975, and these can be used when appropriate.

a. Janbu Slope Stability Charts. The stability chart solutions developed by Nilmar Janbu (Janbu 1954, Duncan & Buchignani, 1975) are particularly useful in coastal engineering since they accommodate many common situations as listed above and have proven to give good estimates of stability in such cases. The Janbu stability charts, like most other, are based on the development of dimensionless mathematical parameters which permit a direct solution of the stability problem as opposed to the time consuming iterative approach used in the analytical methods. The Ordinary Method of Slices, which presumes a cylindrical sliding surface was used to develop the Janbu Charts.

In using the Janbu Stability Charts it is most important to understand the basic assumptions on which the solution is based. The following assumptions are listed by Janbu (1954, pp 16).

a. The potential sliding surface is cylindrical

b. The analysis is two dimensional.

c. The shear strength is governed by Coulomb's Equation:

$$s = c + \sigma \tan \phi \qquad (2)$$

wherein: s = shear strength of soil

c = cohesion

$\tan\phi$ = friction coefficient

ϕ = angle of internal friction

σ = stress normal to sliding surface

d. The shear strength is fully mobilized at every point along the sliding surface, except in zones containing tension cracks.

e. The factor of safety is defined as the ratio between the available shear strength along the critical sliding surface and the shear stress necessary for equilibrium along the same surface.

The terms used in this section of the report are used as defined in ASTM D653-67.

The procedural steps for solving for bank stability by these charts is included in Figure 11 with all the terms defined in Figure 12a. The Janbu Charts are included in Figures 12b, 12c, 13, and 14 with Figure 12 giving the basic stability and Figures 13 and 14 providing correction factors for specific conditions. The Charts are predicated on a failure circle passing through the toe of slope. This is in accord with the general experience for a bank with a soil where $\varphi > o$. The example problem in Figure 15 shows the application of the method to a typical bank slope.

b. Factors Affecting Bank Stability. Having presented a method of stability analysis for a simple homogenous soil profile, it is important to recognize the ramifications to bank stability of the possible deviations from that ideal. In most cases quantitative analysis

(1) Estimate the location of the critical failure circle assuming the circle passes through the toe of slope.

(2) Calculate by a weighted average the value of cohesion (c) and friction coefficient (tan φ) using the applicable portion of failure arc as the weighting factor.

(3) Calculate the term P_d:

$$P_d = \frac{\gamma H + q - \gamma_w H_w}{\mu_q \mu_w \mu_t}$$

with terms defined as in Figure 12a below, and,

μ_q = surcharge correction factor (Figure 13a)

$\mu_q = 1$, for no surcharge

μ_w = surmergence correction factor (Figure 13b)

$\mu_w = 1$, for no submergence

μ_t = tension crack correction factor (Figure 14)

$\mu_t = 1$, for no tension crack

(4) Calculate the term P_e:

$$P_e = \frac{\gamma H + q - \gamma_w H'_w}{\mu_q \mu'_w}$$

where H'_w is the height of water within the slope

μ'_w = seepage correction factor (Figure 13b)

$\mu'_w = 1$ for no seepage.

For rapdily applied surcharge, stability prior to consolidation is given by $q = 0$; $\mu_q = 1$

(5) Calculate the dimensionless parameter $\lambda_{c\varphi}$:

$$\lambda_{c\varphi} = \frac{P_e \tan\varphi}{c}$$

for $c = 0$; $\lambda_{c\varphi}$ is infinite

(6) Enter the chart in Figure 12a with α, and $\lambda_{c\varphi}$ to obtain N_{cf}.

(7) Calculate the factor of safety:

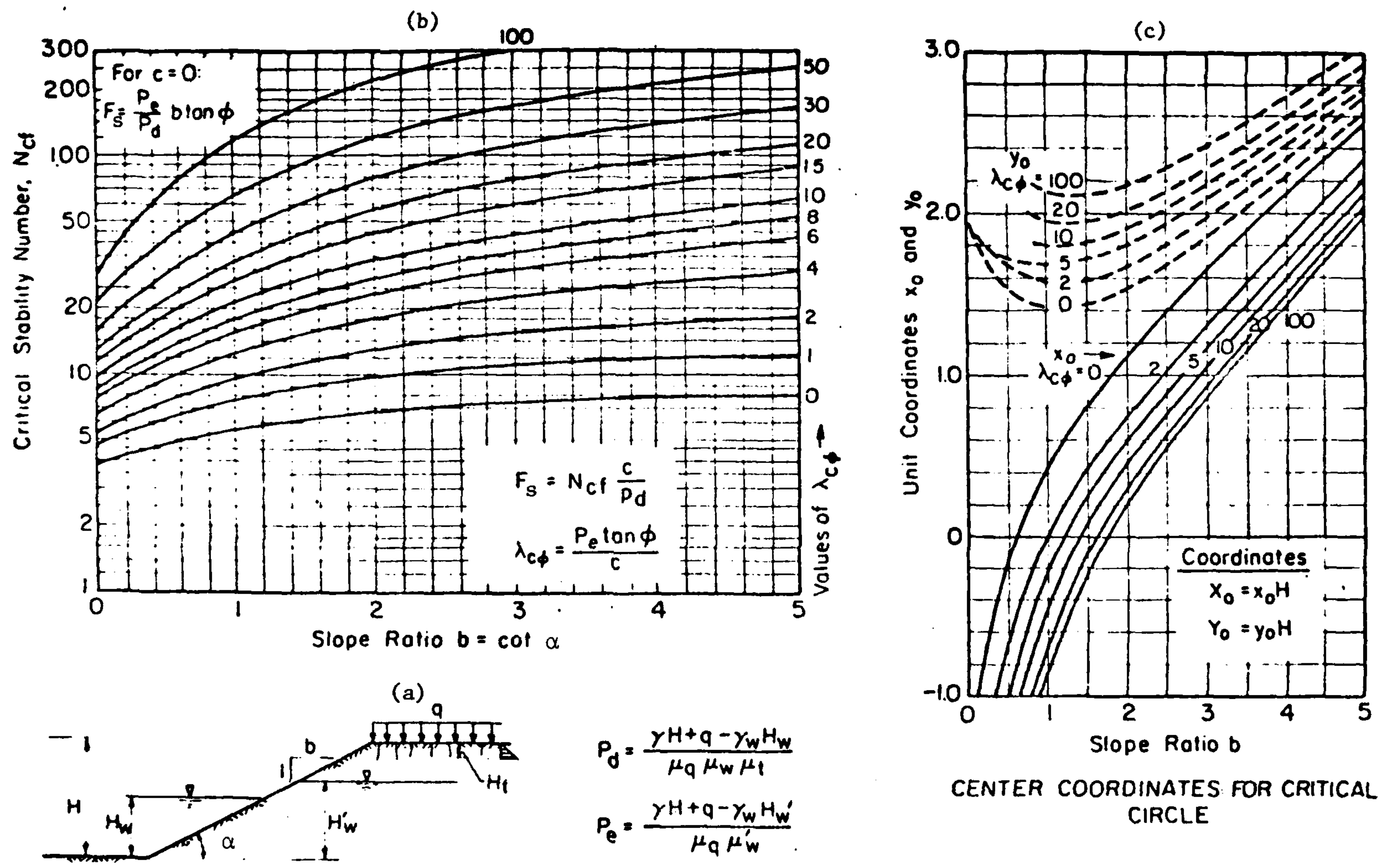
$$F_s = N_{cf} \frac{c}{P_d} \qquad \text{for } c>0$$

if $c = 0$

$$F_S = \frac{P_e}{P_d} b \tan\varphi = \frac{P_e}{P_d} \frac{\tan\varphi}{\tan\alpha}$$

(8) Determine the actual location of the critical circle using the Figure 12b and compare to the assumed circle location.

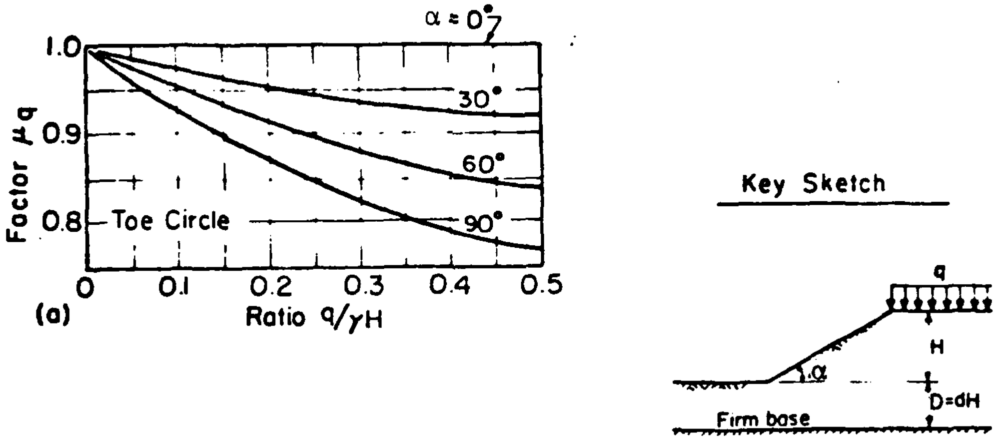
Figure 11. Solution steps for slope stability analysis using Janbu Charts.



(In formula for P_e take $q = 0$, $\mu_q = 1$ for unconsolidated condition)

Figure 12. Slope stability charts for $\phi > 0$ soils. (from Duncan & Buchignani, 1975)

(a) REDUCTION FACTORS FOR SURCHARGE



REDUCTION FACTORS FOR SUBMERGENCE (μ_w) AND SEEPAGE (μ'_w)

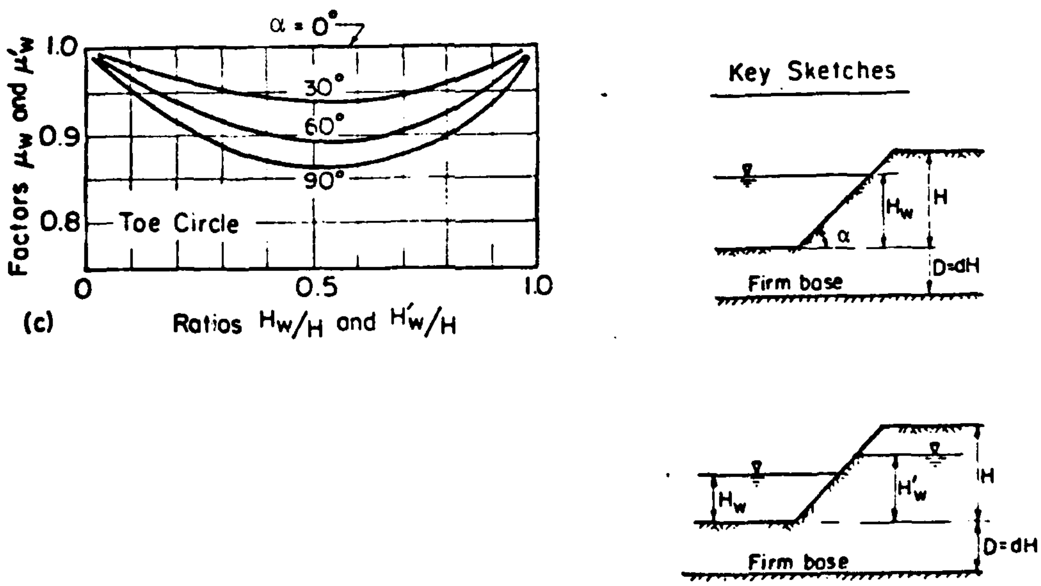
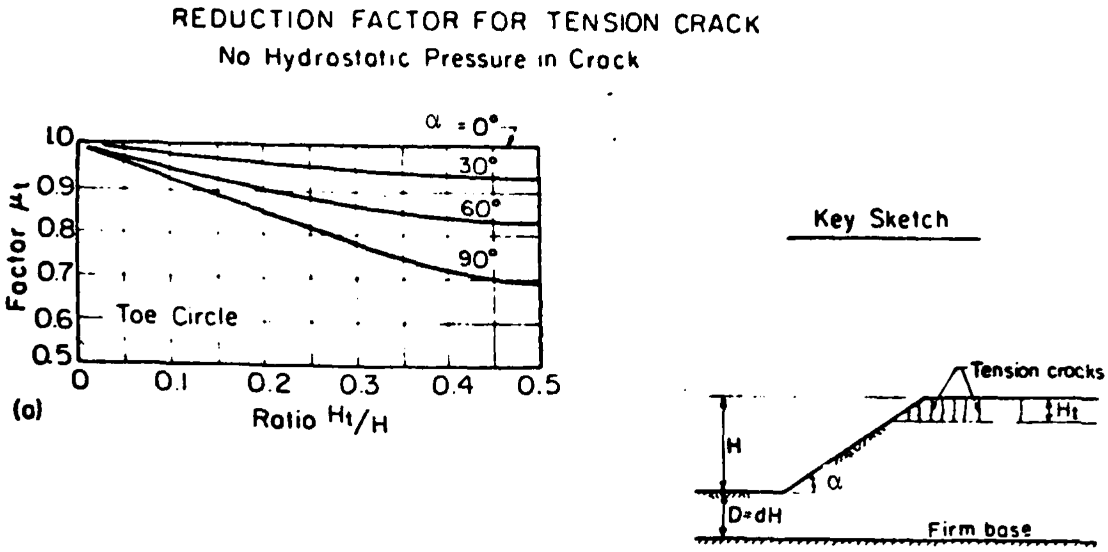


Figure 13. Reduction factors for slope stability charts for $\phi > 0$ soils. (from Duncan & Buchignani, 1975)



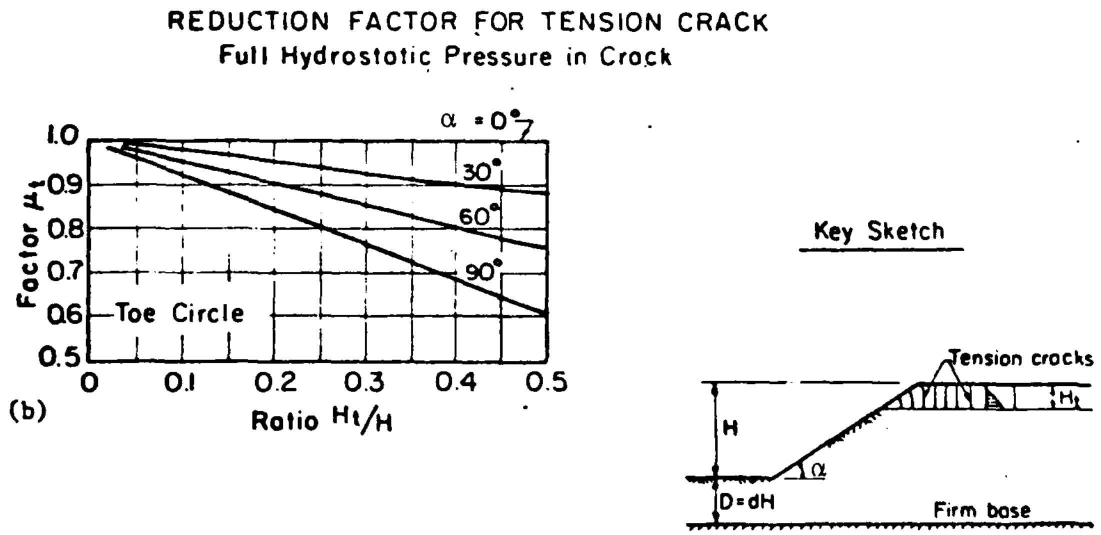


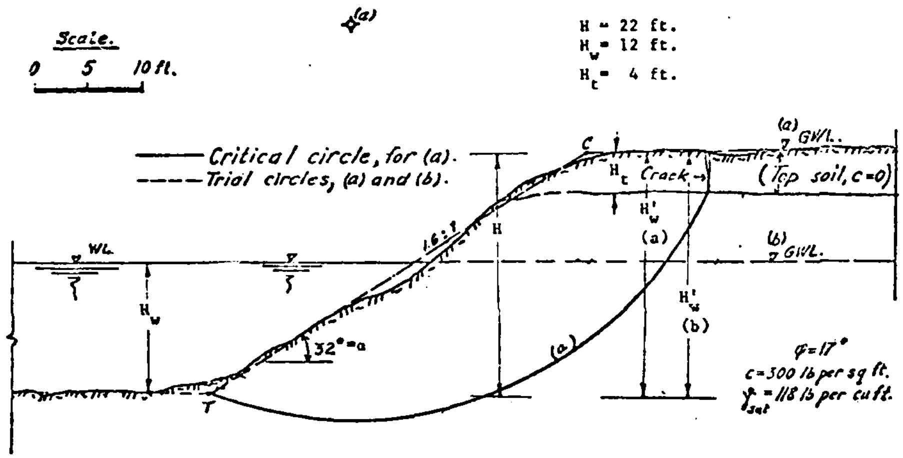
Figure 14. Reduction factors for slope stability charts for $\phi > 0$ soils. (from Duncan & Buchignani, 1975)

Example

Solve for two limiting conditions:

(a) wet weather case, full saturation: $H_w = H = 22$ ft.

(b) dry weather case: $H_w = 12$ ft.



Case (a). from Figure 14: $\frac{H_t}{H} = 0.18,\ \alpha = 32° \longrightarrow \mu_t = 0.95$

from Figure 13b: $\frac{H_w}{H} = 0.55,\ \alpha = 32° \longrightarrow \mu_w = 0.92$

since $q = 0 \longrightarrow$ Figure 13a $\longrightarrow \mu_q = 1.0$

$\therefore\ \mu_d = \mu_w \mu_t \mu_q = 0.872$

and: $P_d = \frac{\gamma_{sat} H + q - \gamma_w H_w}{\mu_d} = \frac{118 \times 22 + 0 - 62.5 \times 12}{0.872} = 2120 \frac{\text{lb}}{\text{ft}^2}$

where: $\mu_e = \mu_q \mu'_w = 1$

$H'_w = H = 22$ ft.

Solving for the dimensionless parameter, $\lambda_{c\varphi}$:

$$\lambda_{c\varphi} = \frac{P_e \tan\varphi}{c} = \frac{1220 \times 0.306}{300} = 1.25$$

Entering Figure 12b: $[\lambda_{c\varphi}, \alpha] \longrightarrow N_{cf} = 9.6$

and:

$$F_s = \frac{N_{cf}\, c}{P_d} = \frac{9.6 \times 300}{2120} = \underline{\underline{1.36}}$$

location of the critical slip circle center

$X_o = 0.62$ $Y_o = 1.54$

$x = X_o H = 13.6$ ft. $y = Y_o H = 34$ ft.

This is circle (a) in Figure 15.

Case (b): from Figure 14b: $\frac{H_t}{H} = \frac{H}{22} = .18,\ \alpha = 32° \longrightarrow \mu_t = .95$

H_w same $\therefore\ \mu_w = .92$

and: $P_d = 2120 \frac{\text{lb}}{\text{ft}^2}$

$H'_w = H_w = 12$ ft. $\therefore\ \mu'_w = \mu_w = .92$

$P_e = \frac{118 \times 22 + 0 - 62.5 \times 12}{.92} = 2010 \frac{\text{lb}}{\text{ft}^2}$

$\sigma_{c\varphi} = \frac{2010 \times .306}{2120} = 2.05$

$N_{cf} = 11.4$

then: $F_s = \frac{11.4 \times 300}{2120} = \underline{\underline{1.61}}$

$X_o = 0.50$ $Y_o = 1.60$

$x = 11$ ft. $y = 35.2$ ft

circle (b) plots very close to circle (a)

Figure 15. Slope stability example problem (after Janbu, 1954).

of the effects of these changed conditions are very difficult, however, careful consideration can give a qualitative idea of their effects. Turnbull et al (1966) points out that the relationship between site geology and bank stability is multifaceted and frequently difficult to decipher. The following paragraphs detail briefly some of the basic relationships which should be considered.

(1) Site Geology. Frequently sedimentary deposits contain definite changes in soil types which can be considered as discrete horizontal layers or strata. Such layering may be due to the changes in deposition pattern during the lay down period with sizes of soil particles deposited dependent on the flow velocity of the water, or to the weathering patterns at a given site. When such soil layers exist in or close under a stream bank or channel bank they can greatly effect both the hydraulic erosion and the slope stability of the bank. Whenever a soft layer of clay lays close to the surface under a bank the critical failure surface will generally pass through this layer. In such cases, the failure surface will deviate significantly from circular and the stability should be checked by a wedge analysis or other method of analysis suitable for an irregular failure surface.

Perhaps the most dramatic effect of layering is when a cohesionless strata within or under a bank liquefies resulting in a flow slide. Liquefaction of a cohesionless soil occurs whenever any sudden rise in pore pressure causes a temporary loss of soil strength through elimination of the force carried by intergranular contact, a force called effective stress or strength. Such a condition may be caused by a sudden shock i.e., an earthquake or a blast. However, even when full liquefaction does not occur, any condition which causes a

large rise in pore pressure, such as rapid water level changes in the river, will cause a reduction in the effective soil strength and may result in a mudslide.

In order to evaluate the effects of boat waves on erosion, it is necessary to understand the bank erosion that occurs due to natural currents and waves. Figure 16 shows the scenario of higher bank erosion that Turnbull, et al., (1966) found on the Lower Mississippi River. "First the river erodes a deep pool in its thalweg. An oversteepening occurs at the toe of the bank slope resulting in a subaqueous bank failure." The third step occurs only when the lower slope failure leaves the upper bank unstable. In such cases, the upper bank may experience a shear faulure along a definite surface or a flow failure due to liquefaction. Erosion, such as this, can be occurring simultaneously with wave induced erosion and complicates the problem of establishing cause. Figure 17 shows several cases where the site geology influences the bank stability and the rate and character of erosion. Before any evaluation of bank erosion can proceed, some knowledge of the site geology is necessary.

Layers of cohesionless soil exposed at the bank face may also contribute to erosion even when not directly attacked by waves or currents. Such layers frequently serve as conduits for groundwater which seeps out and down the bank face causing increased erosion. Also, if pore pressures are raised in such layers a loss of shear strength and slope stability may result. Where cohesionless layers are roughly horizontal surface water from above many enter them via tension cracks caused by a small slope slippage. This saturation of cohesionless soils may then lead to a strength loss and a general failure of the

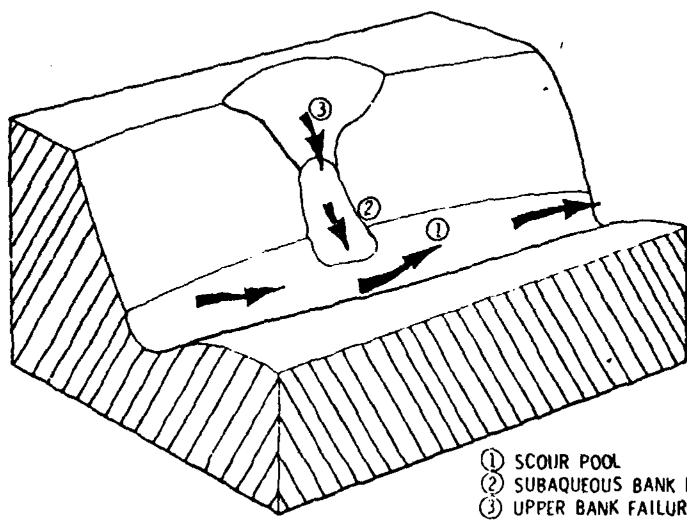


Figure 16. The process of riverbank erosion in sediments of the alluvial valley. (from Turnbull, et al., 1966)

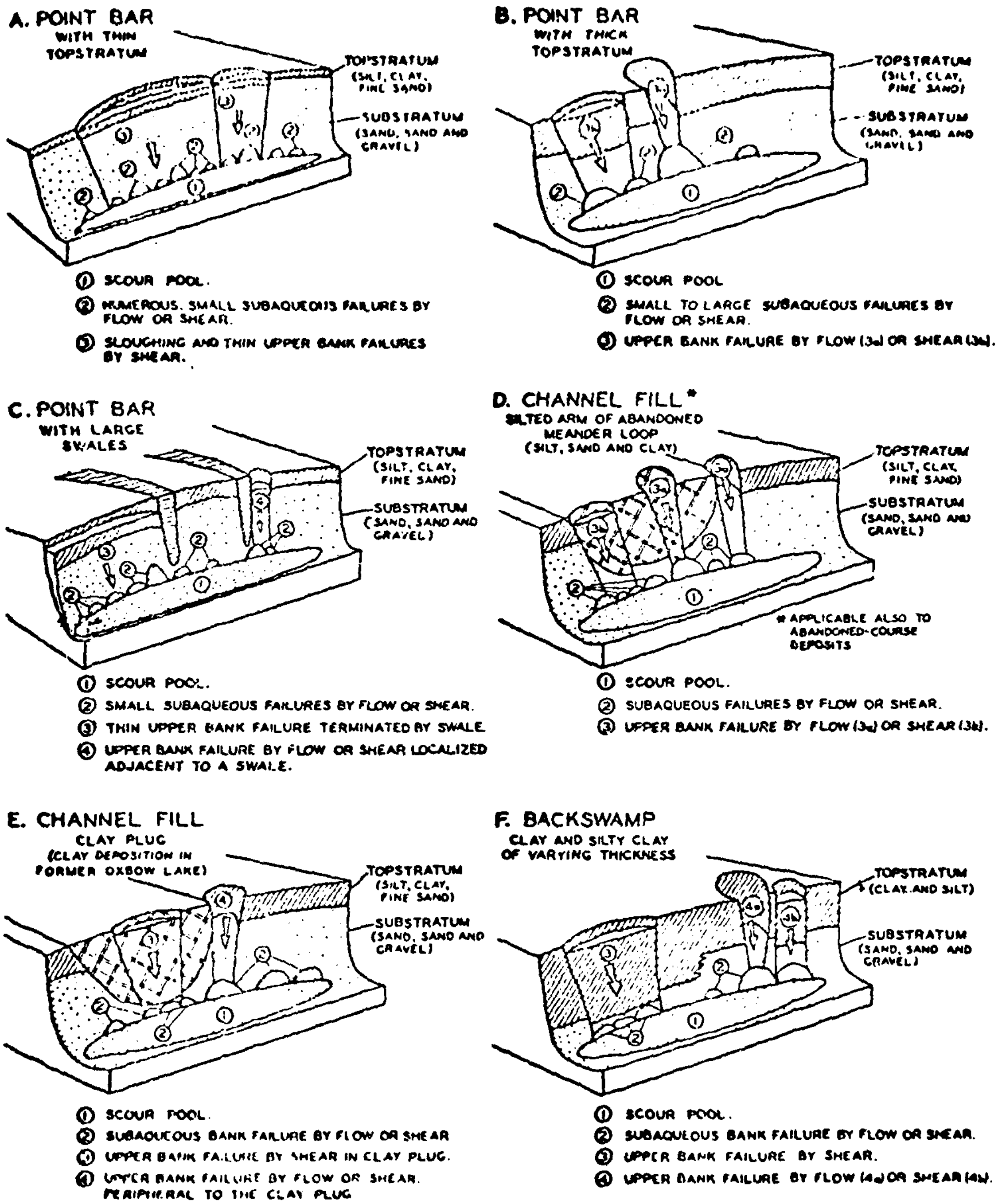


Figure 17. Influence of geology of riverbank soils on the mechanics of bank failure in the alluvial valley of the lower Mississippi River. (from Krinitzsky, 1965)

bank. Seeping groundwater at the bankface may pipe material out of a layer, undermining the soil above until a slide occurs. The above scenario demonstrates the complexity of bank stability and its tie to site geology.

(2) Man-made Structures. Man-made factors also influence slope stability. These include placing surcharges on the top of a bank or slope. Surcharges may be buildings, protective devices, or general fill planned for local uses. Surcharges which lead to tension cracks may set up a chain of events that will lead to a failure during a future rain when cracks become filled with water. Artificial steepening of a slope to permit placement of a bulkhead may also contribute to a slope failure.

(3) Changes in Water Content. Changes in water content due to rainfall, run off or high water may cause loss of strength in several ways. An increase in water content may decrease soil strength by causing swelling in clays or a deterioration of weak cementing materials, as well as by reducing the effective shear stress and strength in the soil. However, a sudden drop in the water level of an adjacent water course, called rapid drawdown, may cause more serious instability by raising the actuating forces which tend to create sliding without immediately changing the shear strength.

3. Evaluation of Bank Recession.

As discussed earlier, bank recession occurs as a result of slope failure, erosion, or a combination of the two. If the recession is due to erosion rather than forces within the bank, recession will proceed in cycles of erosion and slope failure. The cycles may continue indefinitely depending on the causes of erosion and the properties of the bank material. Erosion caused by waves may proceed until the

channel is wide enough to allow dissipation of wave effects, the slope is flat enough to dissipate the wave energy from wind or vessels, a more erosion-resistant material is encountered, or the bank is protected by vegetation or structures. When water currents are the predominate cause of erosion and they persist over a long time, the river bank complies to the currents by selective erosion or deposition that may ultimately create meander patterns, channel alignment changes and cross-sectional area changes. The varying cross-sectional shapes and flow patterns make it difficult to establish, without extensive field measurements, the relationship between the critical velocity at a bank and the net cross-sectional velocity. The changes increase or decrease the erosion rates at a given site.

The bank materials engineering properties are the major determinants of the stable slope profile, the rate of erodibility, and the potential for vegetation cover. The bank's stable slope profile can be estimated, as can the erosion rate if the bank is homogeneous and of a soil for which erosion data exists. For the heterogeneous soil profiles generally found on river banks, there are no general computational methods for predicting erosion rates. The intricate changes in river channels introduce additional complexity which makes the forecasting of erosion at riverside sites very tenuous.

In conclusion, while the measurement of the extent of erosion is rather simple, the evaluation of causes of erosion and the prior prediction of its rate of occurrence is made considerably more difficult by the variability of the soil mass. In the case of bank stability

these natural variables can, with careful observation, be understood and qualitatively evaluated, however, the impact of these variables on a bank's erosive character is much more difficult to assess. Consequently the available methods of estimating the total erosional changes in a bank's profile and location are not satisfactorily accurate for pratical use.

III - NATURAL PROCESSES

The natural causes of bank erosion include wind-generated waves, currents, groundwater seepage, surface runoff, and debris and ice in water which may impact and grind against the bank. These natural effects may act alone in causing bank erosion, or may combine with vessel generated waves and currents.

1. Wind-generated Waves

The height, H, and period, T, of wind generated waves are a function of the fetch length, F, the wind speed, U, and the water depth, d. The U.S. Army Corps of Engineers, Coastal Engineering Research Center (1977) provides figures for determining wave height and wave period in shallow water for given values of water depth. Figures 18 thru 27 are for shallow water wave prediction for water depths varying from 5 to 50 feet. Figures for deep water wave prediction are in the referenced publication. The following example illustrates the prediction of the height and period of wind-generated waves.

********************************EXAMPLE**********************************

GIVEN: A navigation channel is located in a bay with a fetch length F=5 miles, and an average depth d=20 feet. The windspeed U=50 miles per hour, and is along the long axis of the bay.

FIND: The height, H, and the wave period, T.

SOLUTION: From Figure 21, where d=20 feet,

H = 3.8 feet

T = 4.0 seconds

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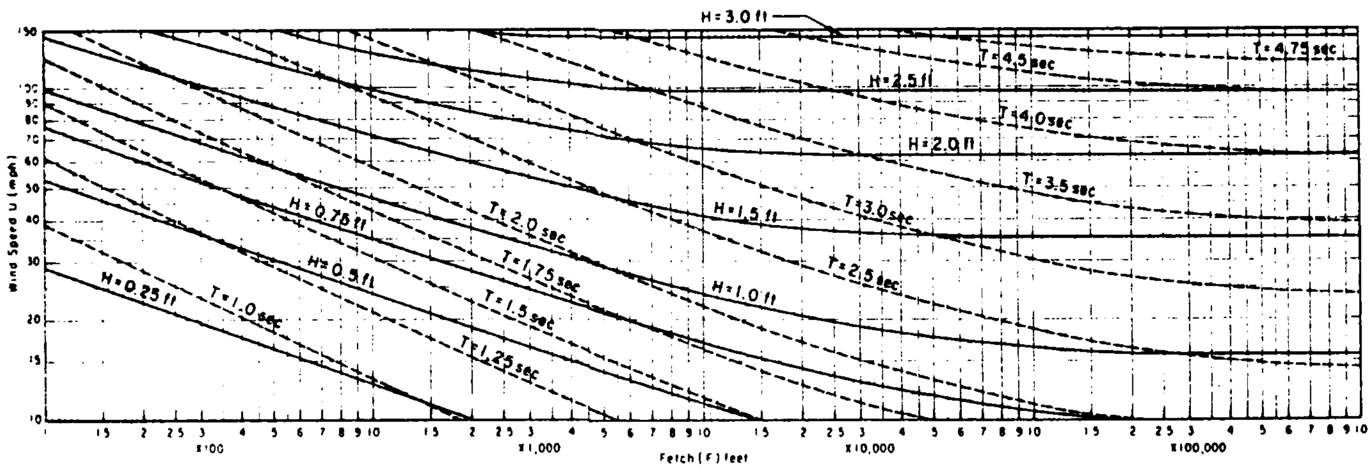


Figure 18. Forecasting curves for shallow-water waves, Water depth = 5 feet.
(from US Army Corps of Engineers, Coastal Engineering Research Center, 1977)

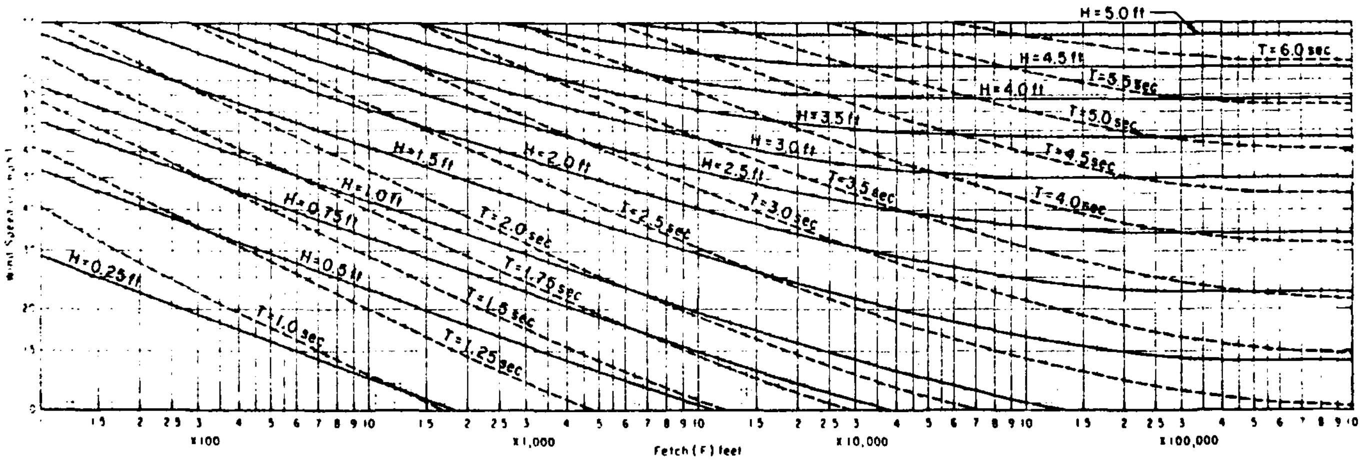


Figure 19. Forecasting curves for shallow-water waves, Water depth = 10 feet.
(from US Army Corps of Engineers, Coastal Engineering Research Center, 1977)

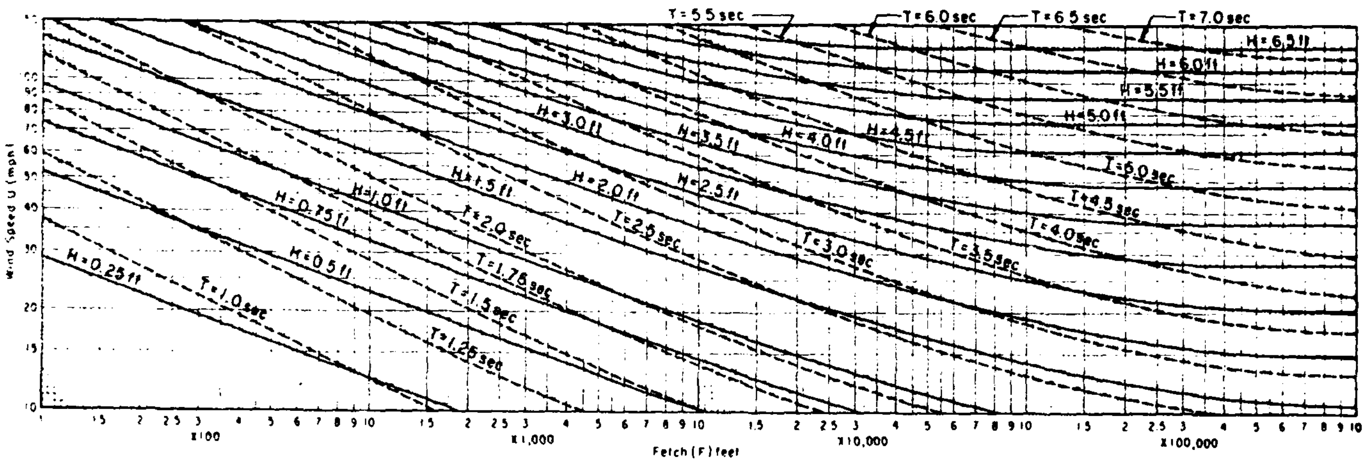


Figure 20. Forecasting curves for shallow-water waves. Water depth = 15 feet.
(from US Army Corps of Engineers, Coastal Engineering Research Center, 1977)

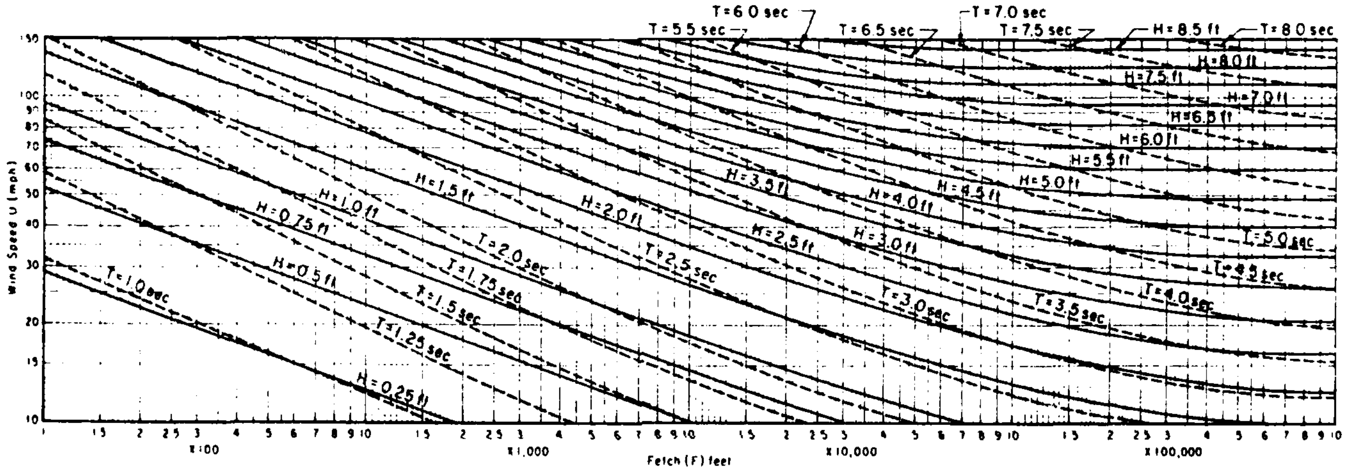


Figure 21. Forecasting curves for shallow-water waves. Water depth = 20 feet.
(from US Army Corps of Engineers, Coastal Engineering Research Center, 1977)

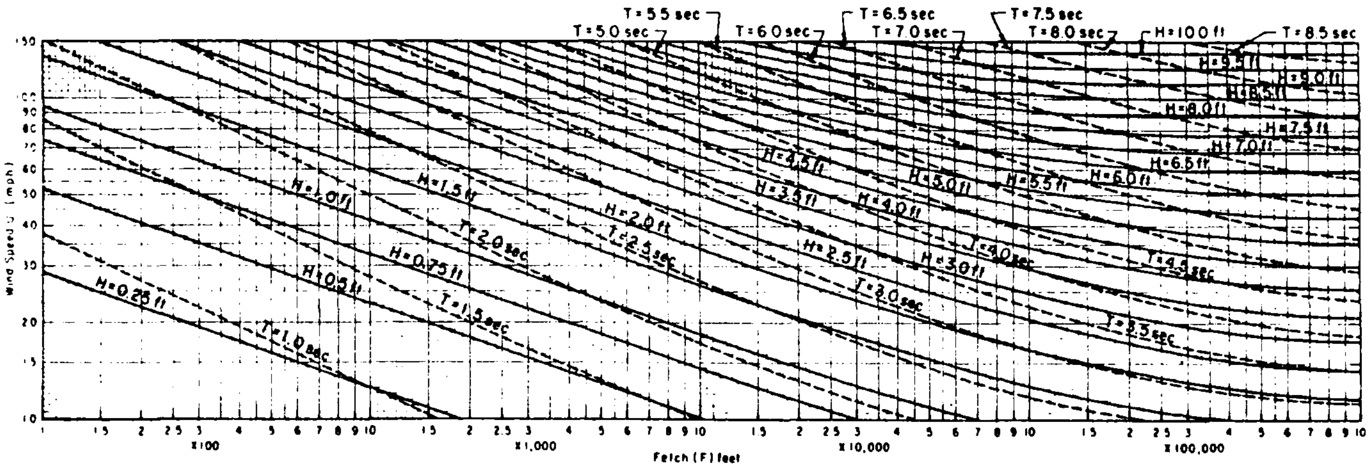


Figure 22. Forecasting curves for shallow-water waves. Water depth = 25 feet.
(from US Army Corps of Engineers, Coastal Engineering Research Center, 1977)

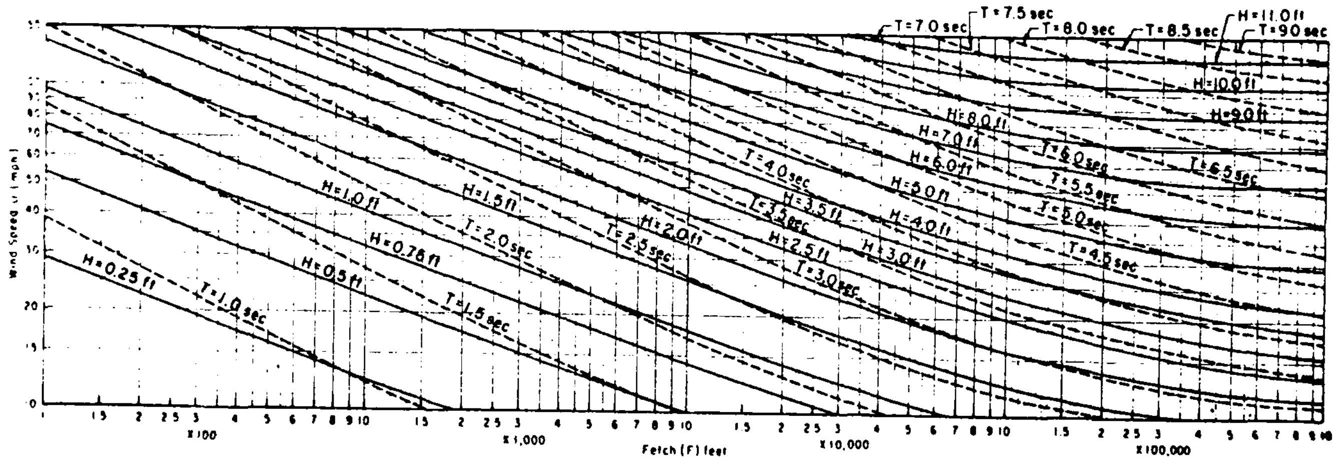


Figure 23. Forecasting curves for shallow-water waves. Water depth = 30 feet. (from US Army Corps of Engineers, Coastal Engineering Research Center, 1977)

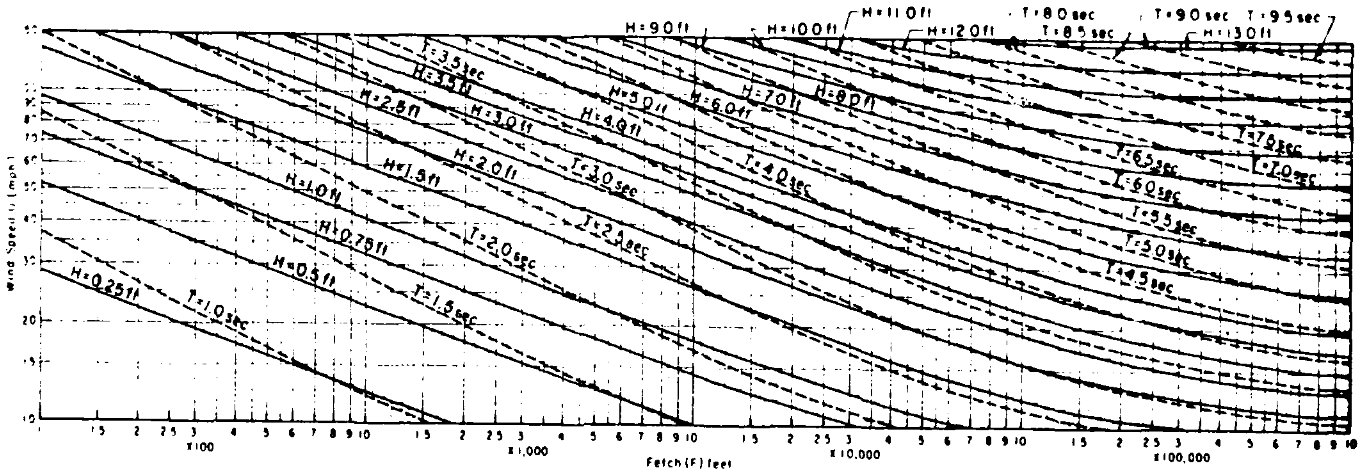


Figure 24. Forecasting curves for shallow-water waves. Water depth = 35 feet.
(from US Army Corps of Engineers, Coastal Engineering Research Center, 1977)

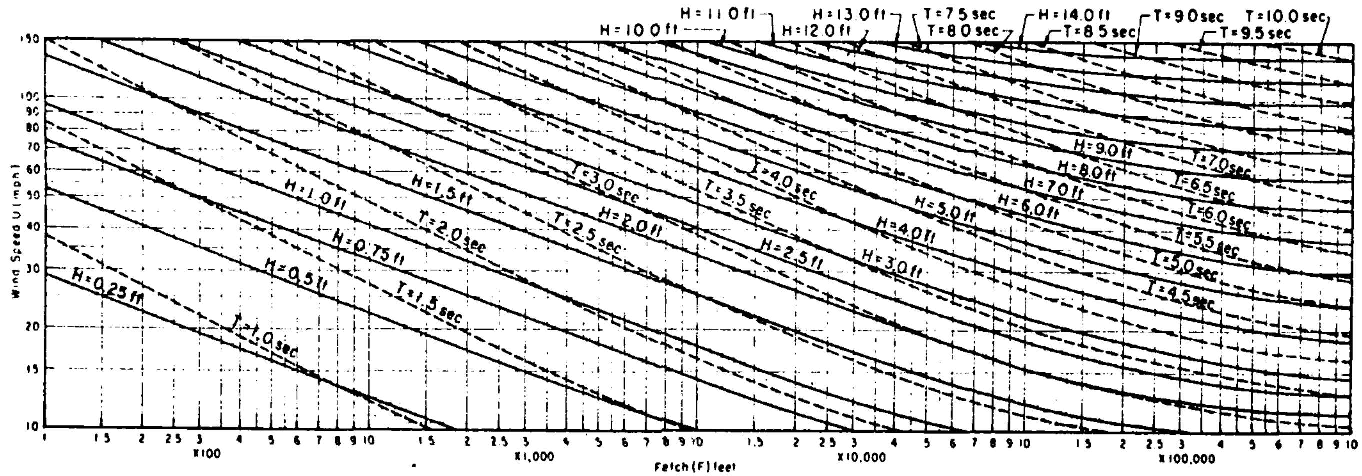


Figure 25. Forecasting curves for shallow-water waves. Water depth = 40 feet. (from US Army Corps of Engineers, Coastal Engineering Research Center, 1977)

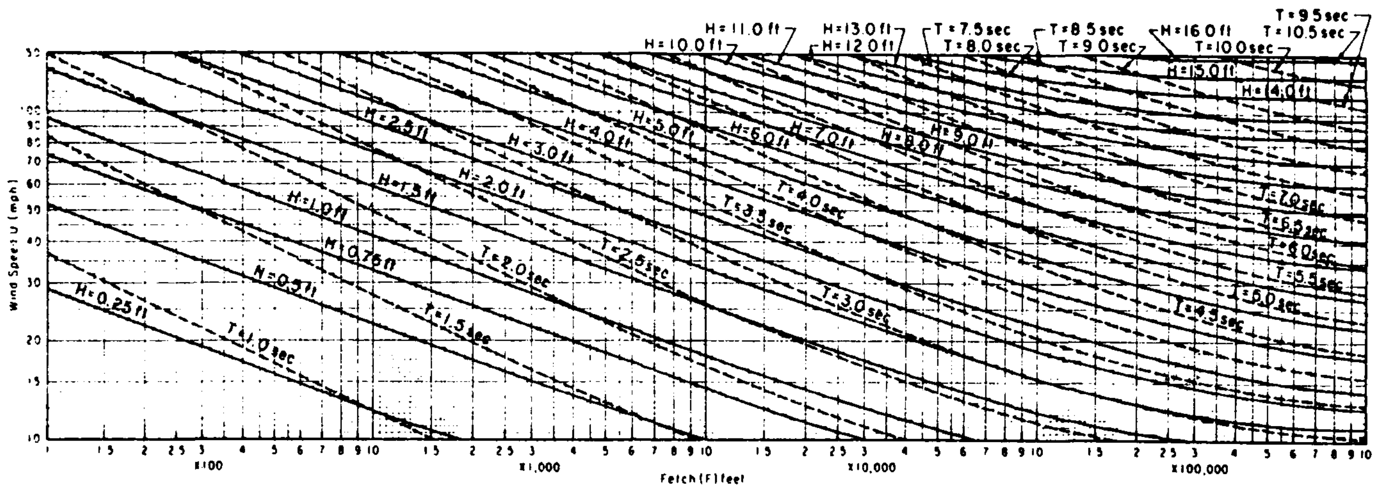


Figure 26. Forecasting curves for shallow-water waves. Water depth = 45 feet. (from US Army Corps of Engineers, Coastal Engineering Research Center, 1977)

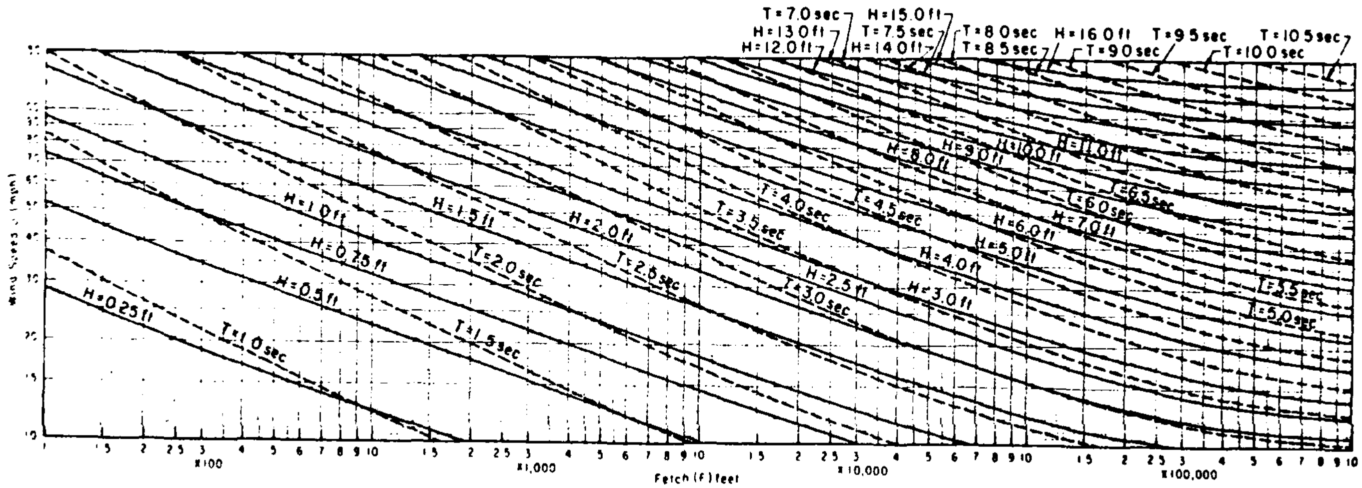


Figure 27. Forecasting curves for shallow-water waves. Water depth = 50 feet.
(from US Army Corps of Engineers, Coastal Engineering Research Center, 1977)

2. Currents

Natural currents present in a waterway may include streamflow and tidal currents. Streamflow will normally be unidirectional, with current velocities slowly varying over long periods of time and dependent upon the variations in runoff in the river basin. Tidal currents reverse direction during the period of the tidal cycle (approximately 12.4 hours for a diurnal tide), and have large variations in magnitude over a short period of time, with periods of slack water.

Tidal current charts are available for a limited number of locations. In general, where bank erosion is occuring, or where there appears to be a potential for bank erosion, it will be necessary to take measurements of current velocities at specific points of interest. If tidal currents are present, measurements should be taken over a period of time equal to at least one tidal cycle. Maximum tidal currents would be expected to occur during a spring tide.

If streamflow is present, measurement of current velocity should be taken at relatively high river stages in order to obtain maximum velocities. As noted in a following report section (Section V, "Particular Areas of Consideration"), in the case of a sinuous channel, the maximum current may occur at different points during different river stages. Therefore, in a sinuous channel, measurements should be taken at several stages to determine the maximum current velocities at any point.

3. Debris and Ice

Currents may carry a variety of debris through a waterway, including dead trees. This debris may impact on the banks, and cause damage to the banks and the bank protections. Hertzberg (1954) discusses damage to some types of bank protection by drift material. The impact of large drift material, such as logs, may initiate damage, and subsequent action of waves and currents may lead to the eventual failure of bank protection and erosion of the banks.

Blocks of ice impacting on channel banks may not be as serious a problem as debris. Waterway banks are normally frozen when ice is present, and are therefore more resitant to damage. However, when water levels rise and fall, a layer of ice on the water surface, which is frozen to vegetation or other bank protection materials, may cause damage by tearing loose vegetation or displacing other materials.

4. Other effects

Where waterway banks are protected by pavement, some damage to the pavement may result from the growth of vegetation. Hertzberg (1954) discusses the deterioration of pavement caused by the growth of vegetation. Vegetation may grow through joints in the pavement, causing spalling, or, in the case of thin pavement, may push through the pavement causing a gradual weakening of the pavement protection.

Natural deterioration may also cause the eventual failure of bank protection. Hertzberg indicates that wooden revetments have a useful life of about twenty years. An additional problem with wooden structures is the theft of wooden members.

Rain water and river water may infiltrate through pavement joints where concrete pavement is used.[3] This may produce cavities under the pavement, and could result in the tilting of pavement slabs.

IV. VESSEL EFFECTS

Vessel effects in a waterway are the result of both waves and currents generated by the ship motion. These waves and currents are determined by a large number of factors including the types of vessels, the numbers of vessels, the vessel speeds, the blockage ratio (the ratio of the submerged portion of the vessel's cross section to the cross section of the waterway), the ratio of the vessel's width to the width of the waterway, the draft of the vessel, the depth of the channel, the geometry of the waterway cross section, the natural currents present in the waterway, the alignment of navigation (the sailing line) with respect to the waterway alignment, and changes in channel alignment.

1. Vessel Waves

Vessel waves include the bow wave, diverging waves propagating at an angle from the sides of the vessel, and the transverse stern wave (see Figure 28). The bow wave depends on a Froude number defined as:

$$\mathbf{F} = \frac{V_s}{(gd)^{1/2}} \qquad (3)$$

where V_s is the vessel speed in the waterway, d is the depth of the channel, and g is gravitational acceleration; and also depends on the relation between ship speed and critical speed. The critical speed of a vessel is discussed by Schofield (1974), and will occur in a constricted channel so that

$$\frac{V_s}{(gd)^{1/2}} = (1 - r_A - 3r_d) \qquad (4)$$

where r_A is the blockage ratio,

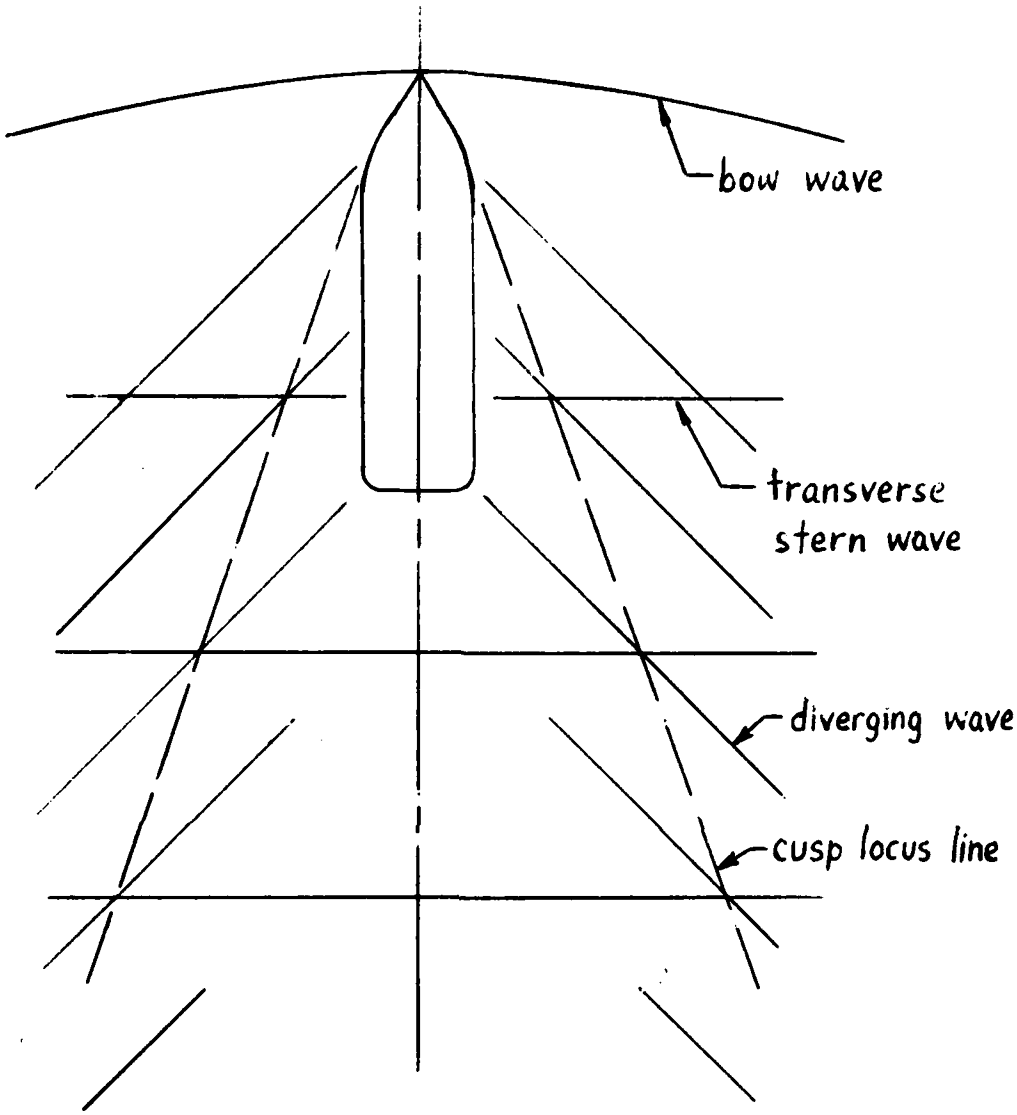


Figure 28. Pattern of waves generated by vessel motion

$$r_d = \frac{d - d_c}{d} \tag{5}$$

and d_c is the minimum depth at the critical condition (see Figure 29). The water level drawdown, Δh, reaches a maximum value when the vessel reaches critical speed. Equation 4 applies to a constricted channel cross section where the return flow produces a critical depth next to the vessel, and in this case the speed of a self-propelled vessel is limited to the value of V_s defined by this equation. Schofield shows that a surge wave (bow wave) is generated ahead of the vessel when the vessel speed reaches the value defined in Equation 4.

Johnson (1968) reported on tests for towed models where the blockage ratio was very small. In this case the bow wave became pronounced when the value of $\mathbf{F}$ given by Equation 3 approached unity. Where $\mathbf{F}=1$, the vessel speed equals the speed of a shallow water wave, so that the vessel moves forward on its bow wave.

Johnson also shows results for diverging waves, as illustrated in Figure 30. Moffit (1968) mapped water surface contours as shown in Figure 31. Figure 31 illustrates the high water elevations along the cusp locus line. Havelock (1908) showed that the cusp locus line is at an angle of $19^{\circ}\ 28'$ from the sailing line where the value of $\mathbf{F}$ given by Equation 3 is less than a value of approximately 0.5.

The transverse stern wave becomes significant when the blockage ratio reaches a significant value. Figure 32 shows transverse stern waves in a laboratory tank where the blockage ratio has a value of 0.17. These waves have the appearance of moving hydraulic jumps, progressing along the channel at the speed of the vessel so that the waves move parallel to the channel bank as turbulent breaking waves. In a relatively constricted channel, these waves will produce the most significant wave energy at the channel bank, and may be particularly high if the cusp locus line intersects the transverse stern wave at the channel bank. Figure 33 shows an example of a boat in a narrow canal. In a restricted channel, such as the one shown in Figure 33, a significant transverse stern wave may occur at relatively low vessel speeds.

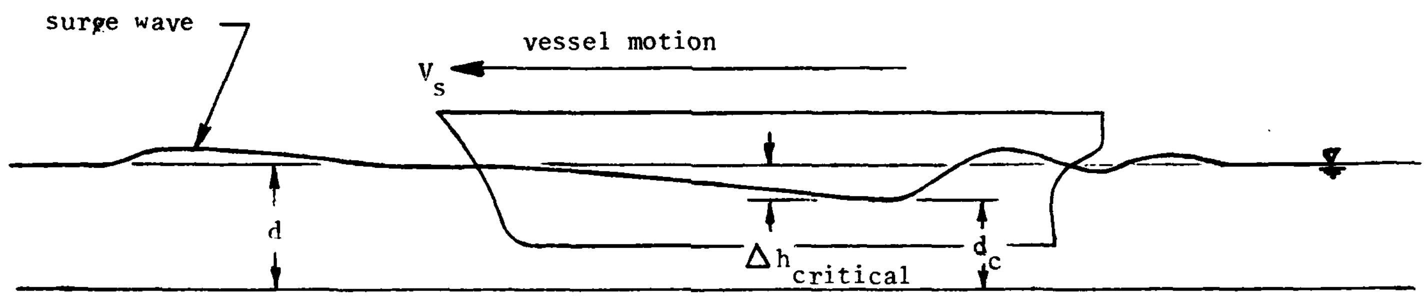


Figure 29. Vessel moving at critical speed

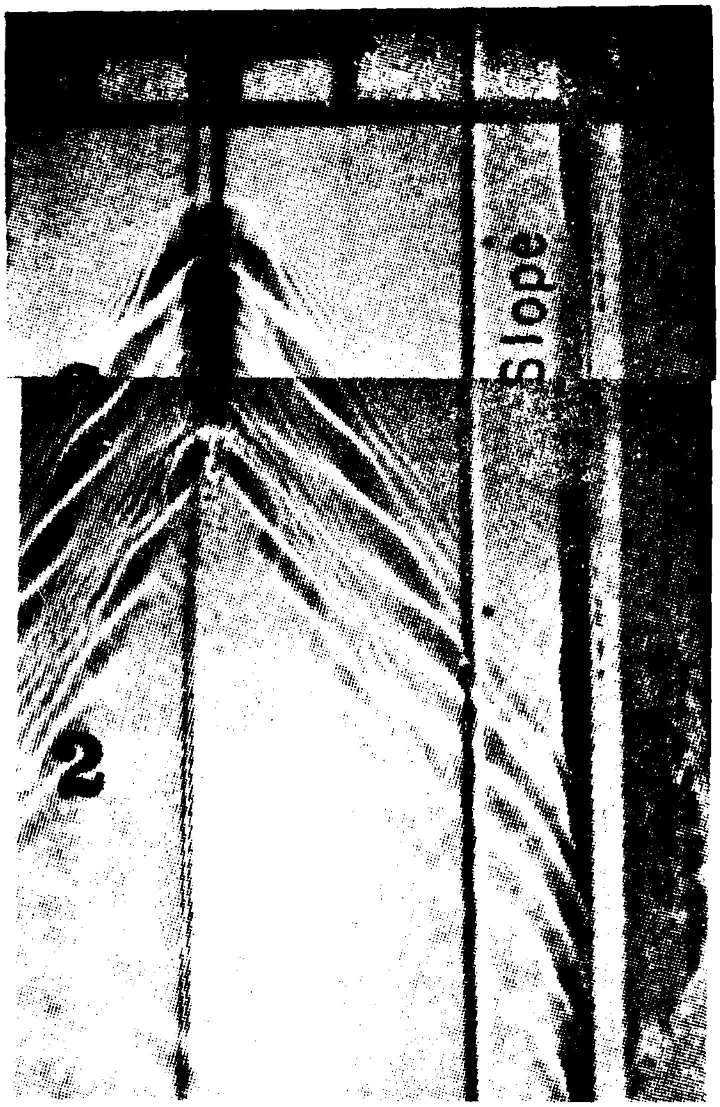


Figure 30. Ship waves in a ripple tank (after Johnson, 1968)

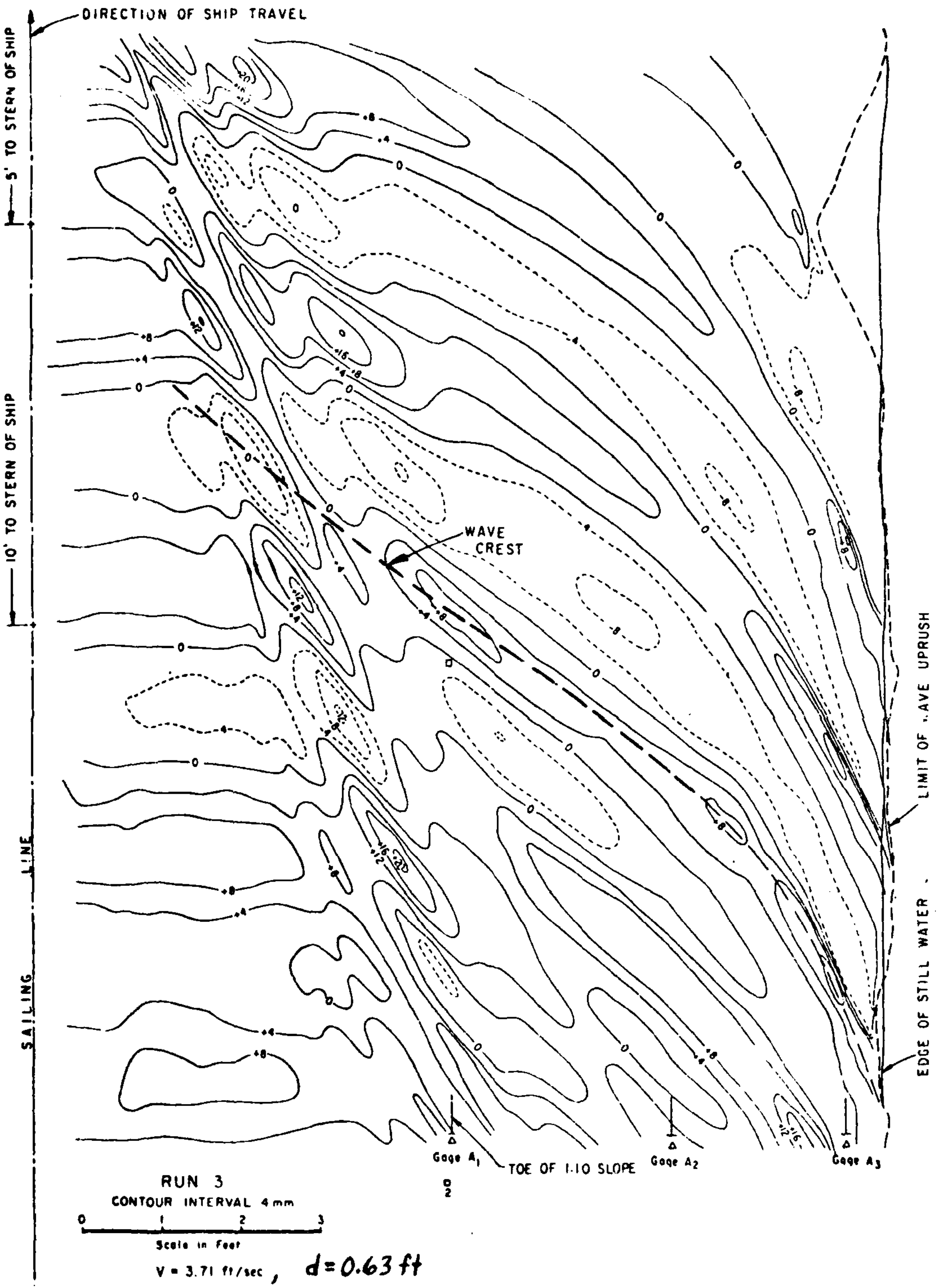
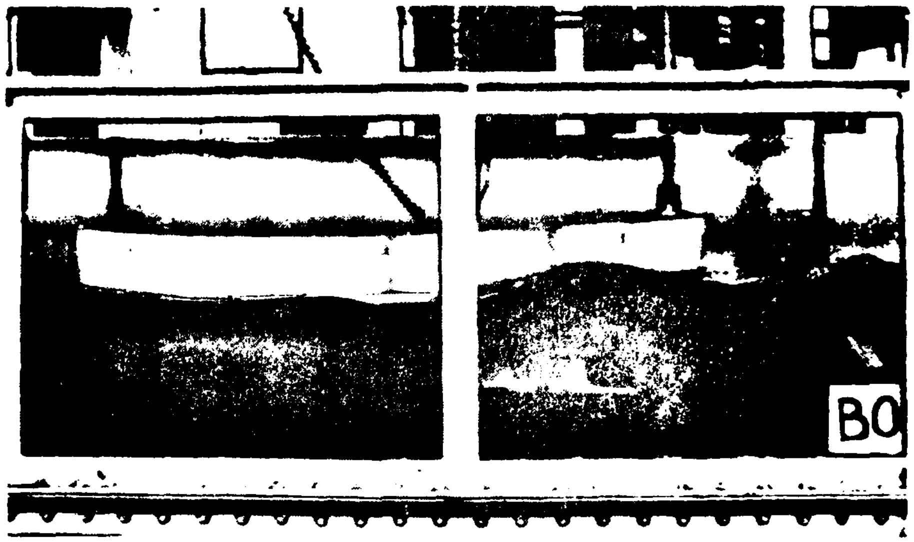
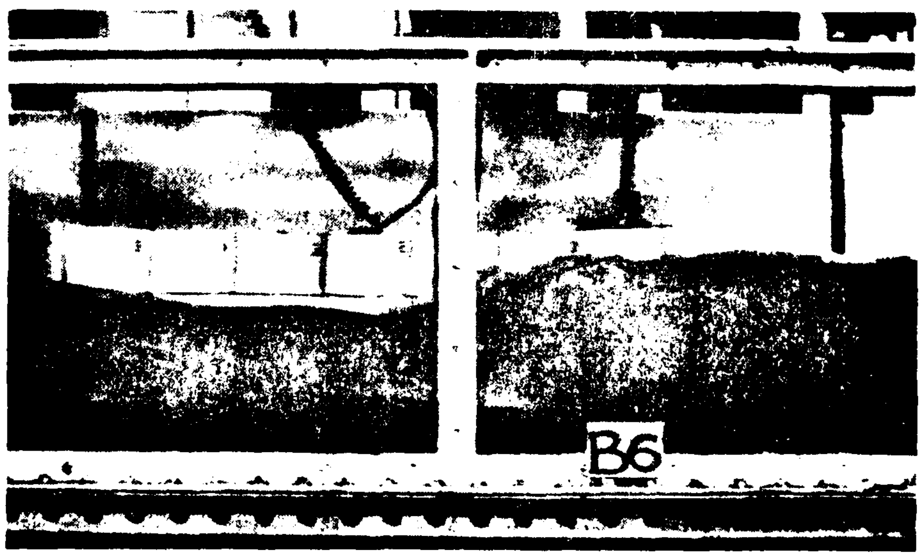


Figure 31. Water surface contours (after Moffit, 1968)



a. Ship model approaching lower critical speed, V = 3.31 fps.



b. Ship model at lower critical speed, V = 3.58 fps.

Figure 32. Trawler hull model, blockage ration 0.17 (from Schofield, 1974)

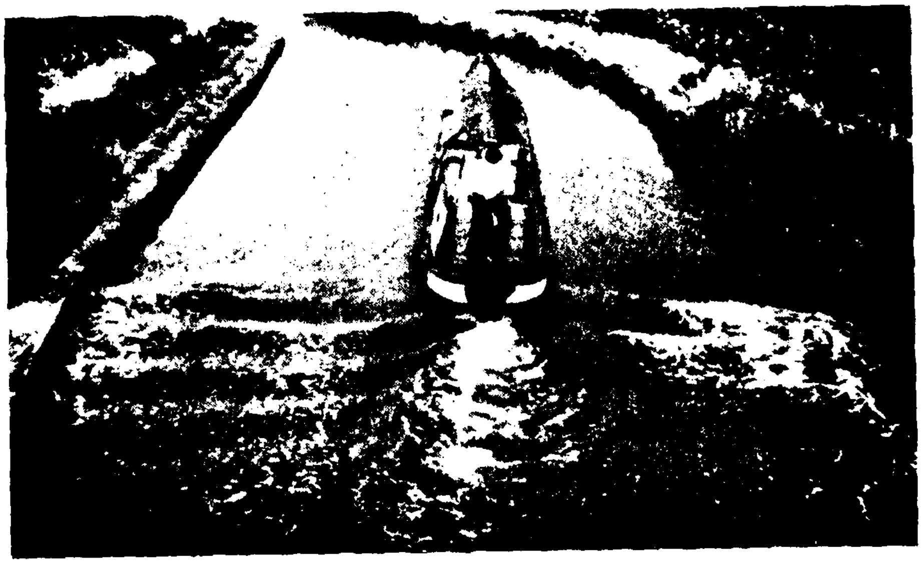


Figure 33. Boat in a narrow canal (from Schofield, 1974)

Dand and White (1978), based on laboratory studies using model tanker hulls, obtained a general expression for the value of water level drawdown, Δh, for ships in canals (shown in Figure 29 for a ship at critical speed). They give this as:

$$\Delta h = 0.39\ (V_s - V_c)^2\ r_A^{\ 1.4} \qquad (6)$$

where V_s and V_c are given in knots, and V_c is the velocity of the ambient current in the canal (positive if in the same direction as vessel motion, negative if in the opposite direction). Dand and White note that this maximum drawdown occurred approximately amidships, and that the water level was approximately constant across the width of the canal (see Figure 33). Similar equations are not available for other hull shapes.

******************************** EXAMPLE ********************************

GIVEN: A ship with a tanker shaped hull is moving through a canal at a speed of approximately 4 knots. There is a one knot current in the opposite direction. The blockage ratio $r_A = 0.1$.

FIND: The water level drawdown, Δh.

SOLUTION:

From Equation 6

$$\Delta h = 0.39\ (V_s - V_c)^2\ r_A^{\ 1.4}$$

$$\Delta h = 0.39\ [4 - (-1)]^2\ (0.1)^{1.4}$$

$$\Delta h = 0.39 \text{ feet}$$

******************************** EXAMPLE ********************************

GIVEN: The same ship as in the example above, with a speed $V_s = 8$ knots, the blockage ratio $r_A = 0.1$, and an ambient current $V_c = 1$ knot in the direction opposite to ship motion.

FIND: The water level drawdown, Δh.

SOLUTION:

From Equation 6

$$\Delta h = 0.39\ (V_s - V_c)^2\ r_A^{\ 1.4}$$

$$\Delta h = 0.39\ [8 - (-1)]^2\ (0.1)^{1.4}$$

$$\Delta h = 1.26 \text{ feet}$$

****

Dand and White also considered a waterway which has a wide horizontal berm, at a shallow depth, between the navigation channel and the bank of the waterway. While they considered the particular case of the Suez Canal, the analysis could also apply to other waterways where the navigation channel is bounded by areas of shallow water. For the case of the Suez Canal, they showed that as the speed of the vessel increased the amount of drawdown over the berm increased. At some vessel speed a weak undular disturbance is initiated over the berm. As the speed becomes higher, the undular disturbance is transformed into a surge wave that travels along the berm. A channel with horizontal berms is illustrated in Figure 34 where d_b is the depth of water over the berm. A general relationship giving the type of wave disturbance, as suggested by Dand and White, is shown in Figure 35 for high values of the blockage ratio.

A number of investigators have measured wave heights generated by various types of vessels in navigation channels where the blockage ratio, r_A, is relatively low. These investigators include Johnson (1958, 1968), Brebner, Helwig, and Carruthers (1966), Sorensen (1967, 1973), and Hay (1968). Sorensen (1973) provides a summary of information on wave heights generated by various types of vessels. That summary is shown in Table 2.

The results in this report are for a single vessel traveling in a waterway with the sailing line approximately parallel to the centerline of the channel. It should be noted that when one vessel passes another vessel in a waterway, the blockage ratio is increased and wave heights will also increase. If a vessel traveling at high speed, in a relatively narrow channel, passes another vessel, the transverse stern waves of the two vessels may be superimposed. This will substantially increase both the water level drawdown and the current effects on the channel banks. Because of the effects of the passing vessel on the maneuverability of the slower vessel, a minimum speed is required for the slower vessel.

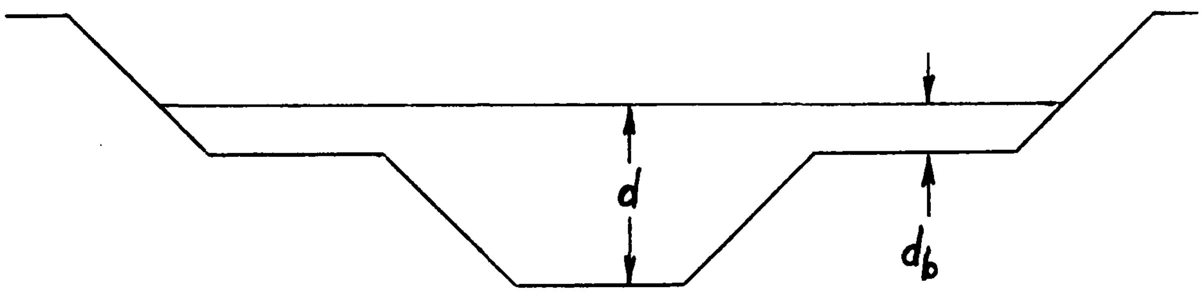


Figure 34. Cross section of channel with berm

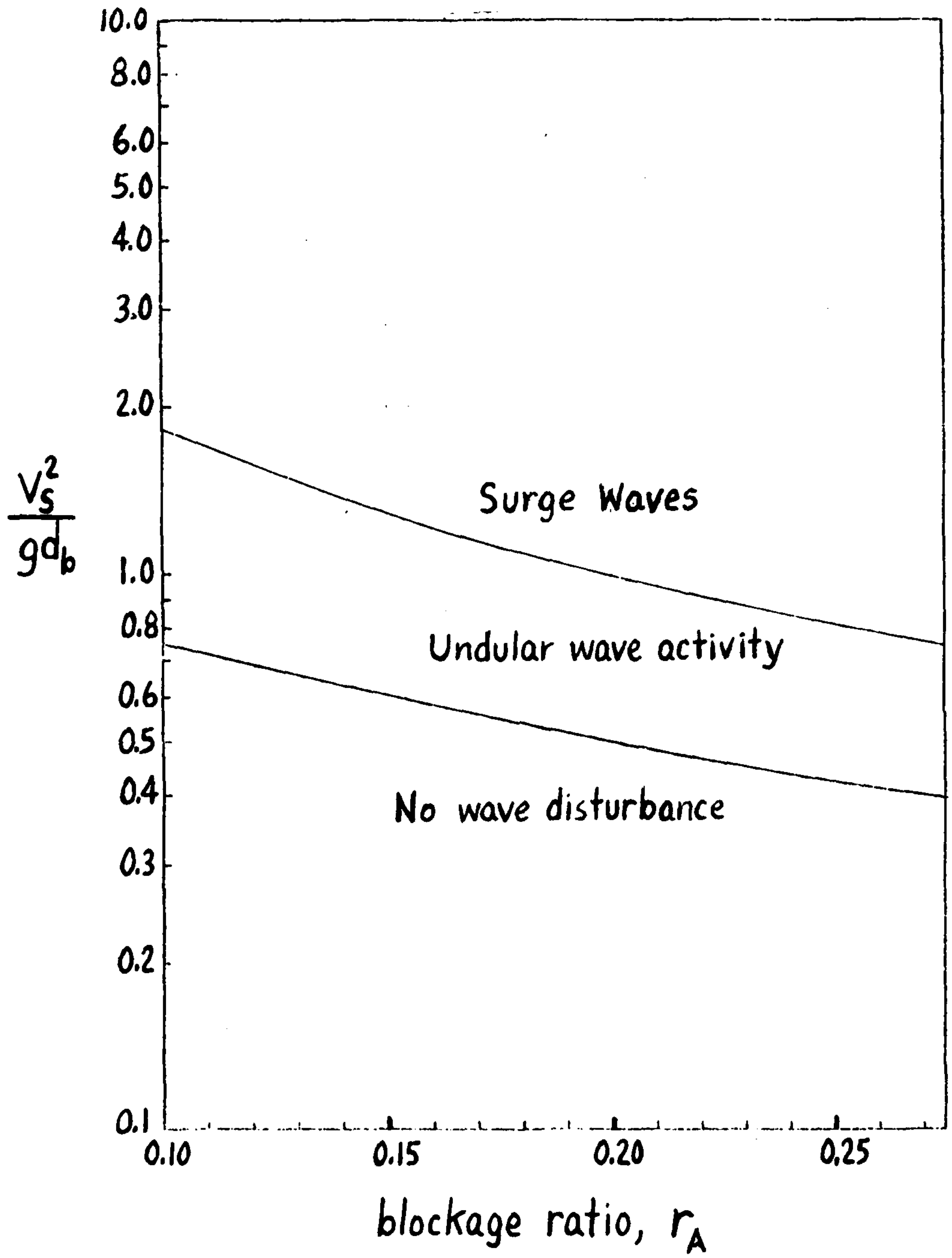


Figure 35. Wave activity associated with drawdown at the berm (after Dand and White, 1978)

TABLE 2. Selected ship-generated wave heights (from Sorensen, 1973)

Vessel type (1)	Length, in feet (meters) (2)	Beam, in feet (meters) (3)	Draft, in feet (meters) (4)	Displacement, in tons (kilograms) (5)	Water depth, in feet (meters) (6)	Speed, in knots (meters per second) (7)	DISTANCE FROM SAILING LINE, IN FEET (METERS): 100 (30.5) H_{max}, in feet (meters) (8)	500 (152.4) H_{max}, in feet (meters) (9)	1,000 (304.8) H_{max}, in feet (meters) (10)
Cabin Cruiser	23 (7.0)	8.3 (2.5)	1.7 (0.5)	3 (2,722)	40 (12.2)	6 (3.1)	0.7 (0.2)	0.4 (0.1)	
						10 (5.1)	1.2 (0.4)	0.8 (0.2)	
Coast Guard Cutter	40 (12.2)	10 (3.0)	3.5 (1.1)	10 (9,072)	38 (11.6)	6 (3.1)	0.6 (0.2)		
						10 (5.1)	1.5 (0.5)	1.0 (0.3)	
						14 (7.2)	2.4 (0.7)		
Tugboat	45 (13.7)	13 (4.0)	6 (1.8)	29 (26,309)	37 (11.3)	6 (3.1)	0.6 (0.2)	0.3 (0.1)	
						10 (5.1)	1.5 (0.5)	0.9 (0.3)	
Reconverted Air-Sea Rescue Vessel	64 (19.5)	12.8 (3.9)	3 (0.9)	35 (31,752)	40 (12.2)	6 (3.1)	0.3 (0.1)		
						10 (5.1)	1.4 (0.4)	0.8 (0.2)	
						14 (7.2)	2.0 (0.6)	1.1 (0.3)	
Fireboat (reconverted tug)	100 (30.5)	28 (8.5)	11 (3.4)	343 (311,170)	39 (11.9)	6 (3.1)	0.4 (0.1)	0.2 (0.1)	
						10 (5.1)	1.7 (0.5)	1.0 (0.3)	
						14 (7.2)	3.1 (0.9)	2.6 (0.8)	
Barge	263 (80.2)	55 (16.8)	14 (4.3)	5,420 (4,917,000)	42 (12.8)	10 (5.1)	1.4 (0.4)	0.7 (0.2)	0.3 (0.1)
Moore Dry Dock Tanker	504 (153.6)	66 (20.1)	28 (8.5)	18,800 (17.1 × 10⁶)	56 (17.1)	14 (7.2)		1.5 (0.5)	1.1 (0.3)
						18 (9.3)		5.2 (1.6)	4.7 (1.4)

2. Currents Associated With Vessel Motion.

The motion of a vessel in a channel causes both a return current and slope-supply flow. Examples of current magnitudes and directions are shown in Figure 36 for a particular example. The return current is caused by the displacement of water in front of the vessel, and flows both under and alongside the vessel from the bow to the stern. The magnitude of the return current is a function of the vessel speed, V_s, and the blockage ratio, r_A, so that the current may become very strong in a narrow channel when the vessel speed is high. As shown in Figure 36, the return current is strongest near the midships section of the vessel, i.e., the section of maximum drawdown shown in Figures 29 and 32, and these effect would combine with the effects from the transverse stern wave in a narrow channel. Equation 6, for the prediction of water level drawdown, gives a qualitative comparison of relative effects at different vessel speeds for a given blockage ratio.

The slope supply flow occurs at a channel section at or behind the stern of the vessel as shown in Figure 36. The slope-supply flow creates currents along the channel bank in the direction of ship motion. While the magnitude of this current is less than the magnitude of the return current, as shown in Figure 36, it acts directly along the waterline at the channel bank and may contribute to bank erosion. As this current velocity is also related to the transverse stern wave, i.e., the water level drawdown, Equation 6 will provide a qualitative comparison of the magnitude of this effect at different vessel speeds, that is, the current becomes much stronger as the drawdown increases.

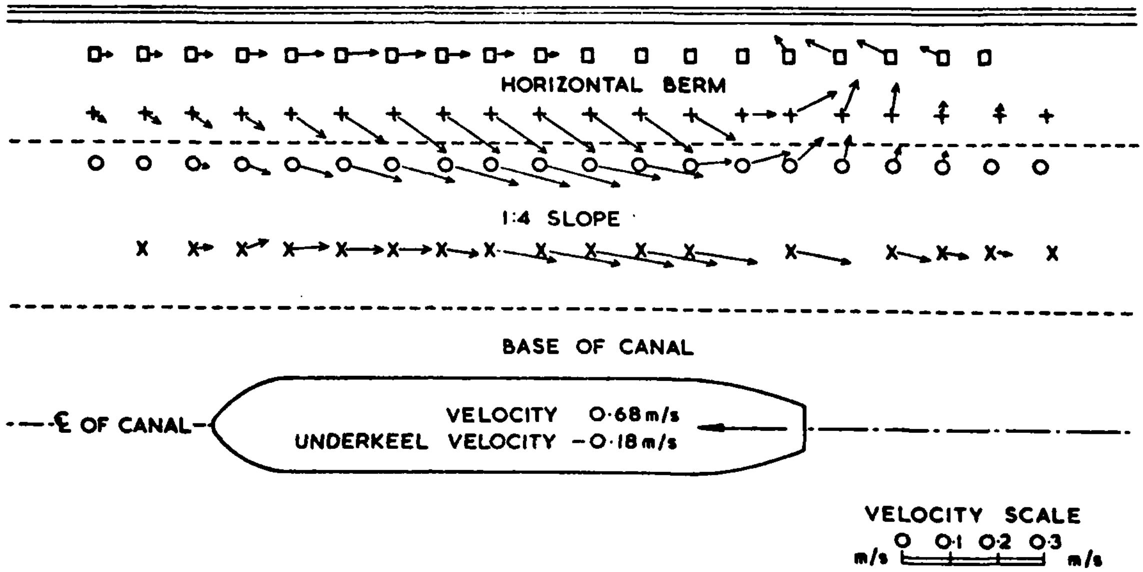


Figure 36. DIRECTION OF RETURN CURRENTS, Suez Canal model (from Dand and White, 1978)

Bouwmeester, et al., (1977) investigated a push-tow traveling near a canal bank. For a water level drawdown of about 1.25 meters (4.1 feet) they observed current velocities near the canal bank as high as 1.5 meters per second (4.9 feet per second) in the direction of vessel motion. They did not report the vessel speed. They observed stone displcement in the direction of vessel motion in an area from 0.5 meters (1.6 feet) above to 2.0 meters (6.6 feet) below the still-water level. They clearly observed displacements caused by the slope-supply flow, often far behind the breaking transverse stern wave.

Bouwmeester, et al., developed an equation for calculating the return current, assuming a uniform return-current velocity around the ship. For a trapezoidal channel cross section as shown in Figure 37, with the hydraulic depth, D, defined as:

$$D = \frac{A}{T} \tag{7}$$

where A is the channel cross-sectional area and T is the width of the water surface, they define a coefficient, K, as:

$$K = \frac{\frac{\Delta h}{D} - \frac{pD}{T}\left(\frac{\Delta h}{D}\right)^2 + r_A}{1 - \frac{\Delta h}{D} + \frac{pD}{T}\left(\frac{\Delta h}{D}\right)^2 - r_A} \tag{8}$$

where p is the cotangent of the channel sideslope as defined in Figure 37. Defining the natural (ambient) current velocity, V_c, as positive in the same direction as vessel motion and negative in the opposite direction as before, they give the following equation for the magnitude of the velocity of the return current, V_R:

$$V_R = K\ (V_s - V_c) - V_c \tag{9}$$

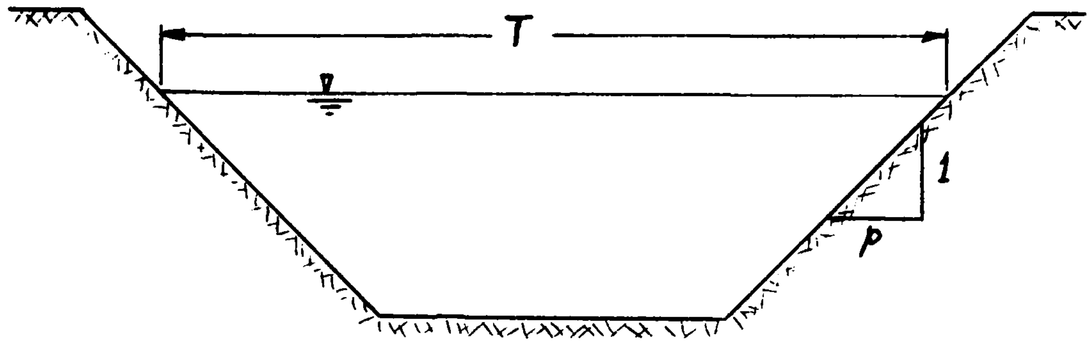


Figure 37. Channel cross section

******************************** EXAMPLE ********************************

GIVEN: A vessel is moving through a channel with a cross section as shown in Figure 37. The channel cross section is 4,000 square feet, the width at the water surface is 360 feet, and the cotangent of the sideslope, p = 3.0. The vessel speed is 8 knots, the blockage ratio is 0.10, and the ambient current is 0.5 knots in the direction opposite to vessel motion.

FIND: The velocity of the return current, V_R.

SOLUTION:

From Equation 6

$$\Delta h = 0.39\ (V_s - V_c)^2 r_A^{1.4}$$

$$\Delta h = 0.39\ [8 - (-0.5)]^2\ (0.1)^{1.4}$$

$$\Delta h = 1.12 \text{ feet}$$

From Equation 7

$$D = A/T = 4{,}000/360 = 11.11 \text{ feet}$$

From Equation 8

$$K = \frac{\frac{\Delta h}{D} - \frac{pD}{T}\left(\frac{\Delta h}{D}\right)^2 + r_A}{1 - \frac{\Delta h}{D} + \frac{pD}{T}\left(\frac{\Delta h}{D}\right)^2 - r_A}$$

$$K = \frac{\frac{1.12}{11.11} - \frac{3x11.11}{360}\left(\frac{1.12}{11.11}\right)^2 + 0.1}{1 - \frac{1.12}{11.11} + \frac{3x11.11}{360}\left(\frac{1.12}{11.11}\right)^2 - 0.1} = 0.25$$

From Equation 9

$$V_R = K\ (V_s - V_c) - V_c$$

$$V_R = 0.25\ [8 - (-0.5)] - (-0.5) = 2.6 \text{ knots (4.4 ft per sec)}$$

****

Using the assumption that there is a uniform return-current velocity around the ship, Equation 9 provides an approximate value for the velocity. Similar estimates for the current velocity of the slope-supply flow have not been developed as a part of this study.

3. <u>Propeller Jet</u>.

An additional effect which may cause erosion is the propeller jet. This is particularly true when vessels are navigating close to a channel bank. Various investigators have reported on this problem, including Balanin and Bykov (1965), Fuehrer and Römisch (1977), and Liou and Herbich (1977). All of these studies relate the velocity, V, of the propeller jet at any point to the initial velocity, V_o, of the jet immediately behind the propeller. As the values of V_o were obtained using coefficients or theoretical developments which are not readily available, reliable quantitative estimates of this value cannot be easily obtained. However, a qualitative analysis, including some use of values obtained theoretically, gives some indication of the expected effects from the propeller jet.

Fuehrer and Römisch (1977) investigated propeller jets of vessels navigating close to a channel bank. They found that the velocities induced by a propeller jet were inversely proportional to a ratio of shaft horsepower to propeller diameter, h_p/D. This relationship is shown in Figure 38, where V_{max} is the maximum velocity on the channel bottom near the bank. It was also shown that the axis of the propeller jet bends towards the bank when the vessel is near the bank. The angle between the jet axis and the sailing line of the vessel is 7 degrees. The rudder angle, and the angle between the channel bank and the sailing line of a vessel maneuvering near the bank, may also direct the propeller jet towards the bank.

The wash from a propeller jet occurs some distance below the water surface, depending on the draft of the vessel, and may undermine bank

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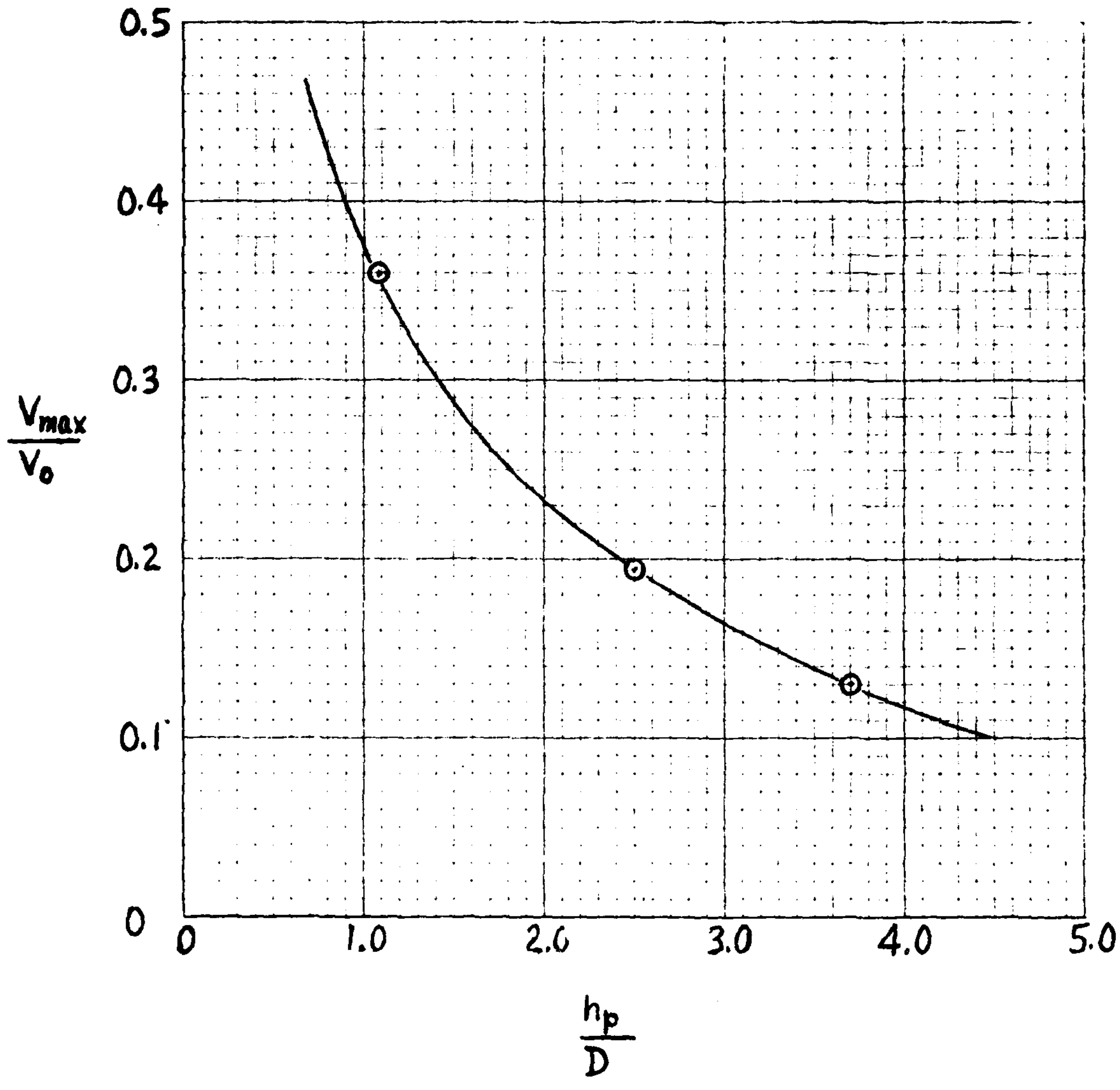

Figure 38. Maximum bottom speed
(from Fuehrer and Römisch, 1977)

protection. The erosion from propeller jets may be especially pronounced in channel bends where barge tows, for example, must maneuver around the bend.

4. Special considerations

In some instances, vessels traveling in waterways may cause damage to banks and bank protection as a result of other effects in addition to waves, currents, and the propeller jet. One effect is the impact of vessels, either deliberately or accidentally, on the channel bank. Where waterways have earthern banks on a moderate slope, vessel operators wishing to go ashore may run the bows of their vessels up on the bank. This may cause some initial damage to the bank slope which, coupled with other erosive forces, tends to flatten the slope. Vessels which accidentally run into a bank may damage revetments or other forms of bank protection, initiating bank erosion.

Damage to the banks of waterways may also result from the practice of mooring vessels by tying them to trees near the banks. This eventually results in the girdling of the trees, as shown in Figure 39, and the dead trees may be pulled into the waterway. This can result in bank erosion where the tree roots are pulled loose, and will also add snags to the channel as a danger to navigation.

Martin and Goede (1935) noted that vessels traveling through ice covered channels may generate a stern transverse wave which causes raising and lowering of the ice layer along the banks. As noted in Section III, "Natural Processes," this movement of the ice layer may tear loose vegetation and displace other bank protection materials.

Figure 39. Trees girdled by mooring lines.

V. PARTICULAR AREAS OF CONSIDERATION

Some areas within a waterway need special consideration when vessel effects are being considered. These include bends in the waterway, changes in a channel cross section, or areas near lock entrances. Changes in channel alignment, e.g., bends in the waterway, may produce strong current velocities in some portions of the cross section, and these stronger current velocities may result in strong erosive forces within the waterway. A change in channel cross section will modify currents and waves traveling in the direction of the channel alignment. In areas such as those near locks, special consideration needs to be given to the effects of prolonged running of vessel engines at a fixed location.

1. Changes in alignment

Keown, et al. (1977) refer to the work of Russell (1967) is discussing changes in alignment. Figure 40 illustrates the effects of a sinuous channel on current velocities. These velocities concentrate near the bank of a waterway and produce a steep bank as shown in Figure 41. The steepening of the bank increases the susceptibility to erosion at a bend in a waterway, and the bank would be more easily eroded by waves and currents generated by passing vessels. Vessels navigating around a bend in a channel may also travel closer to the bank, partly due to the deep water near the bank and partly due to the maneuvering requirements of the vessel. This tends to increase the vessel effects on the bank at the point where the bank is most easily eroded.

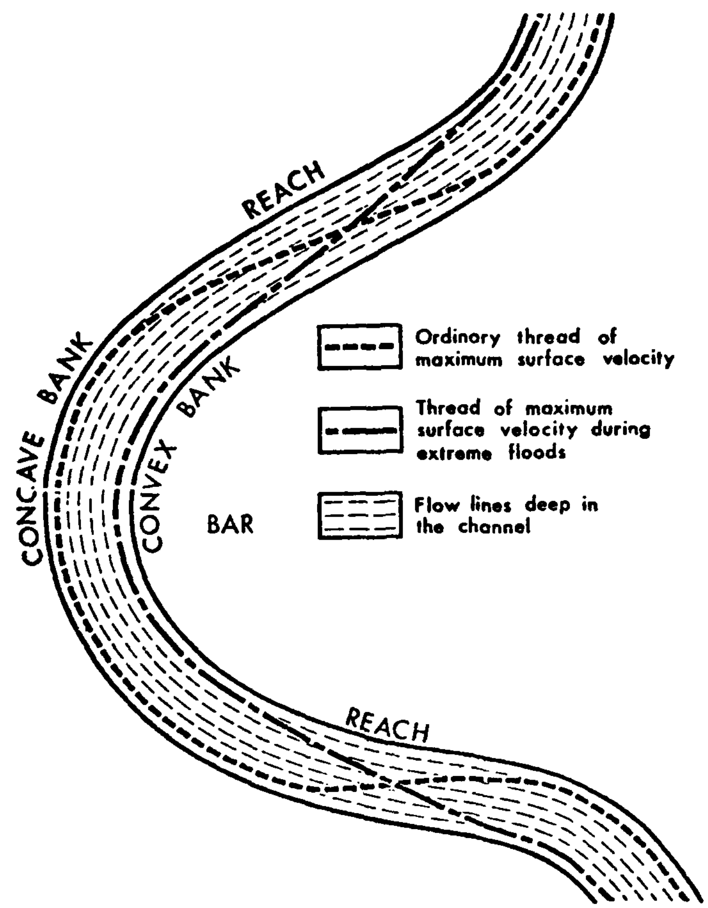

Figure 40. Location of maximum surface velocity in a sinuous river during normal and flood flows (after Russell, 1967)

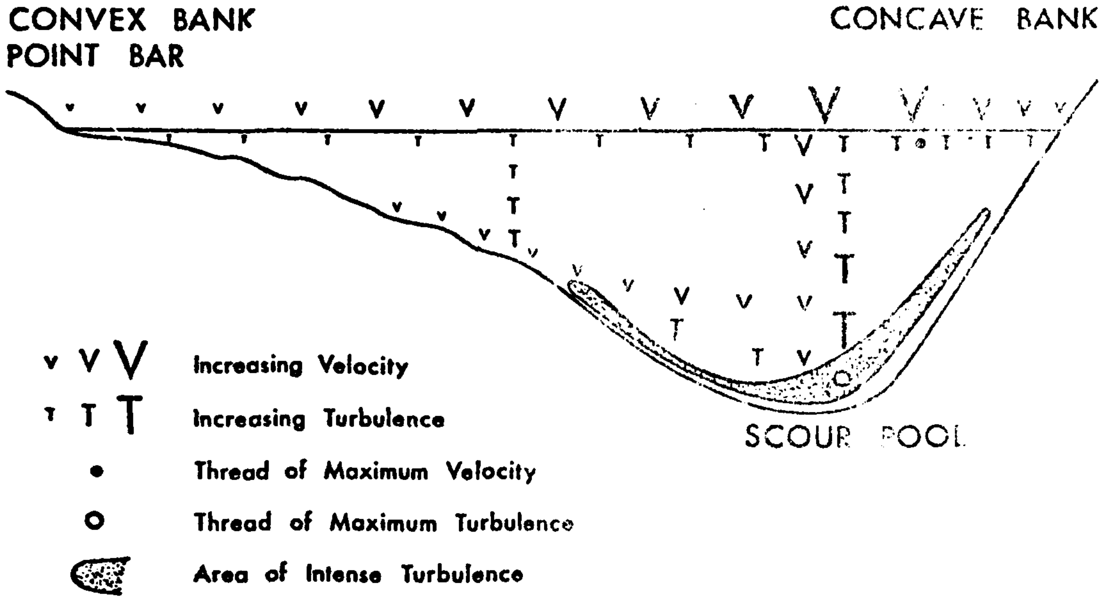

Figure 41. Relative magnitude of velocity and turbulence in a river bend (after Russell, 1967)

2. Changes is cross section

Changes in the cross section of a waterway, or of a navigation channel within a waterway, may cause pronounced changes in waves or currents between the channel sections. The transition zone between channel sections may be particularly susceptible to erosion if strong natural turbulence occurs near the waterway banks because of the transition. In areas where erosion occurs, or in areas where there is concern about potential erosion, the natural currents would need to be investigated on a case by case basis to determine their magnitude.

3. Mooring and queuing areas

As discussed previously in *Section IV*, "Vessel Effects," the wash from a propeller jet may cause bank erosion, particularly when a vessel is close to the waterway bank. This type of bank erosion may be most likely to occur in areas where vessels idle for long periods of time. Areas which should be considered are mooring areas along the bank of a waterway, and entrances to locks or similar areas where vessels may wait in a queue.

In areas where vessels are moored, the mooring maneuvers of the vessels, and the starting and idling of engines before the vessels proceed, result in long periods of engine running near a single locations. Likewise, when vessels are waiting near a lock entrance,

or otherwise waiting to proceed through a reach of a waterway, the vessel's engines will run for some period of time near a single location. In these instances, the wash from the propeller jet may be directed onto a small area of the waterway bank for a sufficient period of time to significantly erode the bank.

VI. BANK PROTECTION

Methods of bank protection range from planting vegetation to constructing covers and structures. Keown, et al. (1977) itemize methods as shown in Table 3. Methods of bank protection presently under investigation are discussed by the US Army Corps of Engineers (1978)

Table 3. Methods of bank protection (after Keown, et al., 1977)

Single-Component Revetment

- Asphalt blocks
- Automobile bodies
- Cellular blocks
- Ceramic riprap
- Concreate blocks
- Rubble
- Sack revetment
- Stone riprap
- Tetrapods
- Trench-fill revetment

Mattresses, Matting, and Pavement Revetment

- Articulated concrete mattresses
- Asphalt pavement
- Bituminous mattresses
- Ceramic mattresses
- Concrete pavement
- Erosion-control matting
- Fascine mattresses
- Gabions
- Log and cable
- Rock-and-wire mattresses
- Synthetic mattresses, matting, and tubing

Mattresses, Matting, and Pavement Revetment (Cont'd)

- Timber-and-brush mattresses
- Used-tire matting

Bulkheads

- Concrete or stone
- Fiber
- Metal
- Timber

Soil Stabilization

- Asphalt (bulk)
- Grout
- Organic mixtures and mulches
- Soil cement
- Thermal control
- Vegetation

River Training Structures

- Cribs
- Dikes (sill, groin, spur, jetty)
- Fences
- Kellner jack field
- Tetrahedron field

Causes of failure of bank protection include natural waves and currents, ice and debris, natural deterioration of material, water infiltration, and vessel effects as discussed in previous sections of the report. Because of the wide variety of bank materials, and the wide range of conditions in different waterways, each particular case of bank failure or potential bank failure must be individually investigated to determine the causes of failure.

In instances where damage to banks is determined to be caused by vessels using the waterway, regulation of vessel traffic may be considered as a means of protecting the banks. Regulating traffic may be difficult because of the variations in vessel hull design, and the resulting variations in vessel wake at a given speed. Speed limits or wake limits (i.e., no wake zones) may be imposed where vessel effects may cause damage, but these limits would require enforcement to be effective.

Variations in water levels, particularly seasonal variations, may allow vessels to approach closer to waterway banks at particular times during the year. This may require delineation of navigation channels within a waterway, and restrictions on navigation close to the banks.

VII. SUMMARY

Recession of waterway banks involves a large number of effects. The physical and chemical nature of the channel's water, the materials forming the bank, and the groundwater in the bank all affect the loss of material to erosion. Changes in the engineering properties of the soil or the height of the groundwater may increase the soil's erodibility by formerly noneroding water currents, wind waves, or vessel wakes. Changes in the channel's water temperature and chemistry may also have an effect. Similar characteristics of the soil and groundwater determine the bank's stability against sliding or the material lost due to bank collapse. Bank instability may be caused by loss of material to erosion, but it may also be caused by changes in the groundwater conditions in the bank, especially an increase in the elevation or seepage rate of water. This report has attempted to briefly catalogue the factors causing bank recession and to summarize the present state of knowledge on evaluating the severity of each causitive factor at a field site.

In order to determine that erosion is the primary source of bank recession, other possible causes must be eliminated. The effect of each factor must be evaluated and weighed against the others, a difficult task requiring considerable expertise, but vital to the success of any attempts to control recession. If, for example, structures are built to control erosion but the bank recession is the result of slope instability due to other causes, the structure may be lost and the whole effort to reduce the problem may prove futile.

If erosion is determined to be a significant factor in bank recession, possible causes of the erosion must be evaluated. These include

currents, wind waves, and vessel wakes. Wind-generated wave heights can be predicted with a reasonable degree of accuracy, and natural current velocities can be measured for individual sites. Some data is available for vessel-generated waves, but the pattern of waves and currents associated with vessel motion is complex. Means of predicting the current velocities set up by a vessel moving in a waterway are not yet well developed. The effectiveness of currents, waves, and wakes in causing erosion can be compared only in terms of the velocity or shear force they generate at the face of the bank. Current velocities at the face can be measured, but relating these velocities to records of velocities measured elsewhere in the channel, if they exist, could require an extensive field research program. Means of accurately converting wave and vessel wake parameters to velocity or shear force terms have not been developed and stand as a major obstacle to the determination of the erosive effects of those phenomena. Even if the relative magnitudes of velocities or forces caused by each source of erosion were known, the critical conditions necessary to initiate erosion, information needed to determine whether the erosion sources are acting alone or in combination, may not be known. Although extensive research has been performed to investigate the erodibility of different types of soil, the lack of data on the erosive characteristics of soils at a field site could be another common obstacle to pinpointing the cause of erosion.

For the specific case of evaluating the erosive effects of vessel wakes, additional data is needed on the currents and waves set up by a vessel moving in a waterway and on the impact of these disturbances on the bank. At present, it may be possible to establish that vessels are

causing erosion at a site, but, in the absence of visual observations of damage caused by a single passing vessel, the effects of a particular vessel on the banks cannot be determined. No computational methods exist for linking a vessel with a chosen hull shape, traveling at a chosen speed in a channel of chosen depth and chosen cross-sectional area and shape with banks of chosen height and materials, to a predicted occurrence of erosion.

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